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ENGINEERING AND DESIGN

DESIGN OF STRUCTURES TO RESIST THE EFFECTS OF ATOMIC WEAPONS

SINGLE-STORY FRAME BUILDINGS

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ENGINEERING AND DESIGN

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SINGLE-STORY FRAME BUILDINGS

INTRODUCTION

7-01 PURPOSE AND SCOPE. This manual is one in a series issued for the guidance of engineers engaged in the design of permanent type military structures required to resist the effects of atomic weapons. It is applicable to all Corps of Engineers activities and installations responsible for the design of military construction.

The material is based on the results of full-scale atomic tests and analytical studies. The problem of designing structures to resist the effects of atomic weapons is new and the methods of solution are still in the development stage. Continuing studies are in progress and supplemental material will be published as it is developed.

The methods and procedures were developed through the collaboration of many consultants and specialists. Much of the basic analytical work was done by the engineering firm of Ammann and Whitney, New York City, under contract with the Chief of Engineers. The Massachusetts Institute of Technology was responsible, under another contract with the Chief of Engineers, for the compilation of material and for the further study and development of design methods and procedures.

It is requested that any errors and deficiencies noted and any suggestions for improvement be transmitted to HQDA (DAEN-MCE-D) WASH DC 20314.

7-02 REFERENCES. Manuals - Corps of Engineers - Engineering and Design, containing interrelated subject matter are listed as follows:

DESIGN OF STRUCTURES TO RESIST THE EFFECTS
OF ATOMIC WEAPONS

EM 1110-345-413	Weapons Effects Data
EM 1110-345-414	Strength of Materials and Structural Elements
EM 1110-345-415	Principles of Dynamic Analysis and Design

EM 1110-345-416 Structural Elements Subjected to Dynamic Loads
EM 1110-345-417 Single-Story Frame Buildings
EM 1110-345-418 Multi-Story Frame Buildings
EM 1110-345-419 Shear Wall Structures
EM 1110-345-420 Arches and Domes
EM 1110-345-421 Buried and Semi-Buried Structures

a. References to Material in Other Manuals of This Series. In t text of this manual references are made to paragraphs, figures, equatio and tables in the other manuals of this series in accordance with the number designations as they appear in these manuals. The first part of designation which precedes either a dash, or a decimal point, identifie particular manual in the series as shown in the table following.

EM	paragraph	figure	equation	ta
1110-345-413	3-	3.	(3.)	
1110-345-414	4-	4.	(4.)	
1110-345-415	5-	5.	(5.)	
1110-345-416	6-	6.	(6.)	
1110-345-417	7-	7.	(7.)	
1110-345-418	8-	8.	(8.)	
1110-345-419	9-	9.	(9.)	
1110-345-420	10-	10.	(10.)	1
1110-345-421	11-	11.	(11.)	1

b. List of Symbols. Definitions of the symbols used throughout t manual series are given in a list following the table of contents in EM 1110-345-413 through EM 1110-345-416.

7-03 RESCISSIONS. (Draft) EM 1110-345-417 (Part XXIII - The Design of Structures to Resist the Effects of Atomic Weapons, Chapter 7 - Single-Story Frame Buildings).

7-04 GENERAL. This manual presents four numerical examples illustratin design procedures and principles given in EM 1110-345-413 through -416. The examples presented are as follows:

(1) The design of a one-story steel frame building, plastic deform tion permitted.

(2) The design of a one-story steel frame building, elastic behavi

(3) The design of a one-story reinforced concrete frame building, plastic deformation permitted.

(4) The design of a one-story reinforced concrete frame building, elastic and elasto-plastic behavior.

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Before illustrating the design of buildings to resist blast loads it is desirable to describe the behavior of the elements of a building frame subjected to blast loads. Accordingly, the first part of this manual is devoted to a description of the response of single-story frame buildings to vertical and lateral blast loads. In the general discussion of frames, it is assumed that the exterior walls are framed vertically between the foundation and the roof so that the columns are loaded laterally only at the top of the frame and are not subject to direct lateral loads such as to cause them to resist these loads by beam action. Columns subjected to directly applied lateral loads are discussed briefly in paragraph 7-12.

7-05 <u>BEHAVIOR OF SINGLE-STORY FRAMES.</u> Single-story frame buildings subjected to lateral blast loads suffer a lateral deflection which is determined by the mass, stiffness, and strength of the structure, the variation of loading with time, the distribution of load on the structure, and the dynamic behavior of the walls and roof. The lateral loads on all walls are transmitted to the roof and the foundations by vertical framing and are carried laterally by the roof slab or roof lateral bracing to the girders of the frame which in turn transmit the load to the columns. The columns carry the lateral loads to the foundation where the reactions are provided by friction and passive pressure forces. The lateral blast loads on the walls are transmitted to the frame girders by either a lateral truss system spanning between frames, or by the roof slab acting as a deep lateral beam.

The resistance of a building frame to lateral loads is a function of the stiffness of the frame columns to relative displacement of the roof and the foundation. The equivalent single-degree-of-freedom dynamic system for the single-story frame is a concentrated mass supported by a massless spring having the lateral resistance properties of the columns (fig. 7.1). The behavior of columns in a frame, and the procedure for designing the columns in single-story buildings, are discussed in paragraphs 7-06 to 7-10. The design of roof girders is covered in paragraph 7-11.

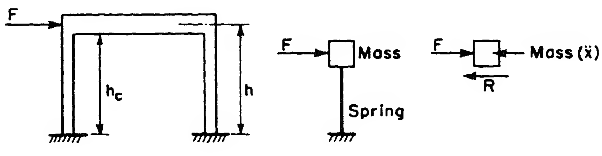


Figure 7.1. Single-story frame and equivalent dynamic system

7-06 SHEAR AND MOMENT RESISTANCE OF COLUMNS. Each column resists th eral motion of the frame through the action of shear forces and bendi moments in the columns as indicated in figure 7.2. The shear resista

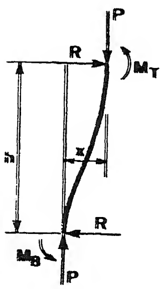


Figure 7.2. Shear resistance and bending moments in a column subjected to lateral displacement and vertical load

in terms of the column bending moments and the axial load is

$$R = \frac{M_T + M_B - Px}{h} \quad \text{(elastic range}$$

$$R = \frac{M_T + M_B - Px}{h_c} \quad \text{(plastic range}$$

where h_c is the clear height of the column as in figure 7.1. In the e range the effect of joint rotation should be included. Thus, using sl deflection equations where θ_B and θ_T are joint rotations at bottom and of the column, respectively, the moments are

$$M_T = \frac{2EI}{h}\left(2\theta_T + \theta_B - \frac{3x}{h}\right)$$

$$M_B = \frac{2EI}{h}\left(2\theta_B + \theta_T - \frac{3x}{h}\right)$$

In the plastic range the top and bottom moments are assumed to b equal to a maximum moment M_D so that

$$R = R_m = \frac{2M_D - Px}{h_c}$$

The value of M_D to be used in equation (7.3) is a variable depen upon the direct stress. The effect of direct stress is discussed in pa graphs 4-07, 4-11b, and 7-07. If there is no direct stress, M_D can be placed by M_P giving

$$R_m = \frac{2M_P}{h_c} \quad (7$$

In equations (7.1) and (7.3), R is a function of M, P, x, and or h_c. In a given design two of the four factors are known; h and h are constants and P is of known variation with time. The remaining t terms are related; i.e., for a given column, M is a function of x a

so that the variation of M and then R may be determined from the variation of x and P.

For a frame with infinitely rigid girders, in the elastic range, the relationship between M and x in any column is obtained from equation (7.2) by setting $\theta_T = \theta_B = 0$.

$$M = M_T = M_B = \frac{6EIx}{h^2} \tag{7.4}$$

With infinitely rigid girders in the elastic range equation (7.1) becomes

$$R = \frac{2M - Px}{h} = \frac{12EIx}{h^3} - \frac{Px}{h} \tag{7.5}$$

Equation (7.5) may be written in the form

$$R = kx - \frac{Px}{h} \tag{7.6}$$

from which the equation for k for one column is

$$k = \frac{12EI}{h^3} \tag{7.7}$$

To obtain the maximum elastic displacement x_e defined by figure 7.3, it is necessary to obtain the maximum or plastic resistance R_m and divide by the spring constant k. For a complete frame with n columns from equation (7.7)

$$k = n \frac{12EI}{h^3} \tag{7.8}$$

and from equation (7.3a), neglecting the entire effect of direct stress,

$$R_m = n \frac{2M_P}{h_c} \tag{7.9}$$

so that

$$x_e = \frac{R_m}{k} = \frac{M_P h^3}{6EIh_c} \tag{7.10}$$

Equations (7.8) and (7.9) apply only when all the columns of the story are identical in strength and stiffness. If this condition is not true, the equations are modified as follows:

$$k = \frac{12E\Sigma I}{h^3} \tag{7.11}$$

$$R_m = \frac{2\Sigma M_P}{h_c} \qquad (7.1$$

where

ΣM_P = sum of plastic column moments in the story, and

ΣI = sum of I values for all columns in the story.

For the case in which the direct stress is considered important, th maximum resistance equation from equation (7.3) becomes

$$R_m = \frac{2M_D n - Px}{h_c} \qquad (7.1$$

and the maximum elastic displacement from equation (7.10) becomes

$$x_e = \frac{M_D h^3}{6EIh_c} - \frac{Pxh^3}{12EInh_c} \qquad (7.1$$

where M_D is a function of P (par. 7-07). Equation (7.12) should be use in numerical analyses where the effect of P and x can be introduced. For preliminary design purposes wherein it is desirable, in order to simplify the computations, to account for the approximate effect of P, the design should be based on

$$R_m = \frac{2M_D n}{h_c} \qquad (7.1$$

7-07 THE EFFECT OF DIRECT STRESS ON COLUMN RESISTANCE. The value of M_D to be used in equations (7.12) and (7.13) is variable dependent upon the direct stress (pars. 4-07 and 4-11). The relationship is different for steel and reinforced concrete. A reinforced concrete section carrying bot direct stress and bending moment has a higher moment-carrying capacity for a limited but important range of axial loads than the same section carryin only bending moment. However, a structural steel section carrying both direct stress and bending moment has a lower moment-carrying capacity than the same section carrying only bending moment.

The limiting elastic deflection when significant axial loads are present is determined from equation (7.13), where M_D is determined from a curve of P_D vs M_D prepared as described in paragraphs 4-07 and 4-11 (see figs. 4.12 and 4.26). As long as the moment M as determined from equation (7.4) and the axial load P together determine a point on the P_D - M_D

graph which is inside the curve, the action of the column is elastic, and the M as calculated is used to determine R from equation (7.5). If the point determined by the values of P and M lies outside the P_D - M_D curve, the action is plastic and the limiting moment M_m is the value of M_D corresponding to the axial load P.

In the preliminary design of reinforced concrete frames, it is desirable to introduce the increased bending strength that results from axial stress in the columns. If this effect is neglected, the preliminary design is generally very conservative. By introducing the direct stress effect a more reasonable column size can be determined.

For steel columns the effect of direct stress is much less important and, in most cases, reasonable results are obtained by neglecting the effect of direct stress in the preliminary column design method of this manual. However, the effect of direct stress is usually considered in making the numerical analysis which is used to check the preliminary design. Column buckling under combined axial load and bending must be prevented in order to maintain the lateral resistance of the frame. In many designs the column section is determined by buckling considerations. For the buckling criteria refer to paragraph 4-07.

In the numerical integration method used to check the preliminary design results, the more comprehensive procedure involves consideration of the individual column direct stresses and their effect on the bending resistance of the individual column. A study has been made to determine whether this precision is necessary. In a series of typical problems, the variation of resistance was determined on two bases: (1) average direct stress equal to the sum of the column loads divided by the sum of the column areas and (2) direct stress determined separately for each column and applied to that column. It has been determined that there is very little loss in accuracy if the average direct stress is used.

7-08 THE EFFECT OF GIRDER FLEXIBILITY ON COLUMN RESISTANCE. In the preliminary design of the columns, the frame response is determined on the basis of the assumption that the joint rotations are negligible. If the girders are designed to act in the elastic range (par. 7-11), the error involved is not large. If all the columns in a story have the same section

and are of equal height, this assumption results in equal moments at th top and bottom of all columns and a linear variation of resistance with displacement up to the plastic resistance. If the assumption of infini stiff girders is not made, the resistance-deflection diagram for a give frame may be determined by a conventional sidesway analysis.

Neglecting the flexibility of the girders results in an overestim of the energy absorption capacity of the frame, thus resulting in under estimates of the displacement of the structure, and the required resist of the columns. Any procedure which reduces the required resistance to value below that needed by the more exact procedure is unconservative. is not desirable to incorporate the flexibility effects into the prelim nary design procedure. The designs obtained by the preliminary design method should be recognized as being slightly unconservative and allowa should be made for this difference by the designer. It is desirable to clude this flexibility effect when the preliminary design is checked by numerical integration procedure.

The recommended procedure for approximating the effect of girder flexibility for use in the numerical integration analysis of single-sto structures is described below. Figure 7.3 presents the form of the

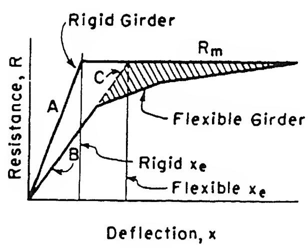


Figure 7.3. Effect of girder flexibility on resistance-deflection diagram of multibay frames

resistance-deflection diagram for a typical mul column frame subject to lateral load only. Lin represents the resistance for infinite girder stiffness. With infinite girder stiffness, pla hinges would develop simultaneously at both end all columns. If the actual girder flexibility considered, the hinges would be found to develo successively as indicated by line B. The recom- mended resistance diagram is line C, an extensio of the initial slope of line B to the intersecti with the line of maximum resistance. The shaded area represents the err introduced. Use of line C will result in the calculated deflections bei smaller than the true deflection. However, the error involved is genera very small. To obtain the effective spring constant k for girder flex bility, it is necessary to determine the slope of line C. This can be

determined by imposing an arbitrary lateral deflection upon the frame and calculating the resistance corresponding to this deflection. The ratio of the resistance to the displacement is $k = R/x$. Paragraph 7-26 illustrates the simple elastic frame analysis that is needed for this determination.

7-09 <u>EFFECT OF LATERAL DEFLECTION ON COLUMN RESISTANCE.</u> From equation (7.1) it may be seen that the resistance is subject to reduction by the combined effect of the lateral deflection and the axial column loads. In single-story frames this effect is small and is neglected in the preliminary design procedures, but it is included in the final numerical analysis.

7-10 <u>DESIGN OF COLUMNS.</u> A general preliminary design procedure for plastic behavior is presented in paragraph 6-11 and for elastic behavior in paragraph 6-12. Details peculiar to application of these methods to single-story column designs are explained below and illustrated in paragraphs 7-24 and 7-34 for steel frames, and paragraphs 7-42 and 7-50 for reinforced concrete frames. The loading used in the preliminary column design is the net lateral blast load as computed from procedures in paragraph 3-09, neglecting the effect of dynamic response of wall panels and other intervening structural elements. The dynamic effects of the mass and structural properties of the walls are accounted for in the final check of the column section by using the dynamic reactions to the front and rear walls in the numerical integration. The equivalent mass concentrated at the top of the column is given by

$$m = \text{total roof mass} + 1/3 \text{ column mass} + 1/3 \text{ wall mass}$$

In the preliminary design of steel columns the frame girders are assumed to be perfectly rigid, and the axial load in the columns is neglected so that the required moments, the spring constant, and the limiting elastic deflection of the columns can be obtained from equations (7.8) and (7.10). From equation (7.9)

$$M_p = \frac{R_m h_c}{2n} \tag{7.15}$$

The cross section required to provide the plastic bending moment resistance M_p is determined from data in paragraph 4-07 for steel columns.

The preliminary design procedure for reinforced concrete columns is different from that for steel because allowances are made for the effect of

direct stress on the bending resistance of the column. After solving
the required M_D from equation (7.14), it is necessary to determine the
dimensions of the cross section. Since M_D is a function of P (par.
it is necessary to use the time average of the total axial load P fo
time interval estimated to be the plastic phase.

Using equations from paragraph 4-11b for eccentrically loaded co
a cross section is selected which will provide the necessary M_D at the
age P .

Having determined the necessary cross section, the next step is
compute k and x_e. For both steel and concrete, $k = 12EIn/h^3$. Howev
from equation (7.10)

$$x_e = \frac{M_P h^3}{6EIh_c} \text{ for steel}$$

$$x_e = \frac{M_D h^3}{6EIh_c} \text{ for concrete}$$

(

These parameters are used in the remainder of the design procedure wit
modification by any load or mass factors because the single-story fram
considered to be directly replaceable by a single-degree system. The
liminary design is then completed in accordance with the steps of eith
paragraph 6-11 or paragraph 6-12.

After the girder is designed, the preliminary column design is v
fied by means of a step-by-step numerical integration procedure. It i
quired that the displacement of the top of the column determined by th
computation be reasonably close to the design displacement determined
consideration of paragraph 6-26.

The more exact numerical analysis includes all the factors which
have been neglected in order to simplify the preliminary procedure. T
are: the effect of girder flexibility, the effect of vertical load ec
centricity, the effect of direct stress, and the effect of the dynamic
response of the wall and roof elements on the lateral and vertical loa
time curves. As discussed in paragraph 7-07, a simplification is poss
by using the average column axial loads instead of considering the in-
dividual columns separately. The effect of girder flexibility is
determined as indicated in paragraph 7-08. Examples of the numerical

integration procedure applied to steel column designs are presented in paragraphs 7-26 and 7-36, and for reinforced concrete columns in paragraphs 7-44 and 7-52.

7-11 <u>DESIGN OF ROOF GIRDERS.</u> The design of roof girders in building frames for blast loads is a difficult problem which is complicated by the time variation of the lateral and vertical blast loads, and the difference in time required for the different girders to reach maximum stress. In general, the maximum frame moments due to the vertical loads develop before those due to the lateral loads. Conventional static loads must be considered in addition to the blast loads. In a building with openings it is possible to have internal pressures of such magnitude as to develop net upward forces on the roof girders.

To obtain the maximum lateral stiffness for the building frame, it is desirable that the roof girders in frames be designed to act elastically. To simplify design procedures, continuous-span beams can be considered as single-span elements with restraints as indicated in figure 7.4. It is recommended that single-span frame girders such as A be designed elastically to carry vertical loads as simply supported beams. Exterior girders such as B should be designed elastically to carry vertical loads as beams fixed at the first interior support and pinned at the exterior support. Interior girders such as C should be designed elastically to carry the vertical loads as beams fixed at both ends. In order to develop the maximum lateral resistance of the frame, it is necessary to design the girders so that the plastic hinges form in the columns. To insure this behavior, the bending strength of the girders at any point must equal the moment at that point due to static and dynamic vertical loads, plus the moment due to lateral motion. The latter is computed by applying to the girder the full plastic hinge moments of all the columns simultaneously. This is conservative because it assumes that all the maximum moments develop at the same time.

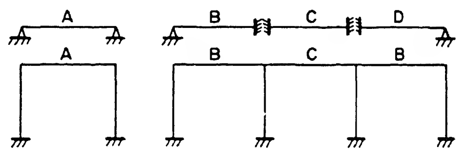


Figure 7.4. Single-span support assumptions for design of frame girders

Consideration of the blast loading on frame girders shows that the front girder is the critical girder in a multibay frame and the critical section is at the first interior support. At this section the frame moment

in the girder should be a fraction of the column maximum moment M_D. Fo 2-bay frame use $1/2\ M_D$. For a 3-bay frame use $2/3\ M_D$ and for a 4-bay frame use $5/8\ M_D$.

The design procedure recommended for girders is an elastic design based upon paragraph 6-12 and consists of the following steps which are lustrated in paragraphs 7-25, 7-35, 7-43, and 7-51. In order to perfor these operations, it is necessary to refer freely to other manuals for formation; viz., to EM 1110-345-413 for overpressure-time variation on roofs, to EM 1110-345-414 for the equations governing the plastic momen capacity of steel and concrete girders, to EM 1110-345-415 for the elas response characteristics of single-degree-of-freedom systems, and to EM 1110-345-416 for the factors which define the equivalent single-degree-freedom system.

Step 1. Obtain the vertical load-time curve for the girder from dynamic reaction of the roof element which the girder supports. Ideali the curve to a form for which dynamic load factors are available.

Step 2. Estimate the dynamic load factor for preliminary size determination.

Step 3. Calculate T_n, the period of vibration of the equivalent single-degree-of-freedom system. T_n is a function of mass m and spri constant k. The equations for k are given in EM 1110-345-416. The mass to be used is the mass which is considered to move with the girder In EM 1110-345-416 consideration is given to beams which have mass vari tions that are triangular in spanwise distribution as well as concentra at local points.

Step 4. Using T_n and T, the time parameter of the loading, obt a new value of dynamic load factor from figures 5.20 and 5.21.

Step 5. With the new dynamic load factor, determine the design m ment required for vertical blast loads. Determine the total moment by adding the static load moments and the proper fraction of the column plastic moment. Select a size to withstand the indicated bending momen in accordance with EM 1110-345-414 requirements.

Step 6. Repeat the cycle, computing T_n, T/T_n, and D.L.F., and ch the section for the revised bending moments including the allowances

described above until satisfactory agreement is realized.

Step 7. If the loading curve cannot be approximated by the idealized shapes used in figures 5.20 and 5.21 it may be necessary to perform a step-by-step numerical integration to check the design for the loading curve.

7-12 FRAMES WITH LATERAL LOADS ON THE COLUMNS. There are certain arrangements of the structural elements in framed buildings which would require that the columns be capable of resisting directly applied transverse loads in addition to providing the resistance to lateral motion of the frame.

This condition is indicated in figures 7.5(a) and (b). Figure 7.5(a) corresponds to the case of a frame building with exterior walls which act as two-way panels. Two edges of the panel load the columns directly, and the other edges transmit load to the roof and the foundation. The portion of the load transmitted to the roof is indicated by F(t). The column loading w(t) is assumed to be uniformly distributed. Figure 7.5(b) corresponds to the case of a frame building with the exterior wall framed horizontally. In this case, there is no concentrated load F(t).

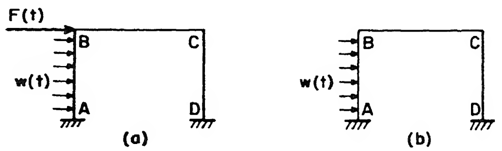


Figure 7.5. Columns subjected to directly applied lateral loads

7-13 LOADS ON FRAME STRUCTURES. The orientation of the blast wave with any element of a building should be assumed to be that which will produce the critical load on that element. The critical load for an exterior wall is produced by a blast wave acting perpendicular to the wall. Frames should be designed for the load produced by the blast moving parallel to the plane of the frame. For roof slabs the critical load is a function of two considerations, the location of the element and the direction of the blast wave. Paragraph 3-05 shows that the overpressures on the central portion of a roof of rectangular plan subjected to a blast wave moving normal to the long axis are less than the overpressures at and near the ends. For the same roof plan, all roof elements would be subjected to the same intensity of overpressure if the direction of the blast wave movement is parallel to the long axis of the building. On the central portions of square roofs the overpressures for all orientations of blast wave are less than at the edges of the roof.

The outside dimensions of the structure must be used in computin blast loading on the structure. This means that the member sizes must assumed initially to obtain the outside dimensions in order to compute loads. A large difference between the assumed dimensions and final de dimensions would require a revision of loads.

7-14 FRAMING ARRANGEMENTS. In the blast resistant designs of this ma there are no radical departures from conventional framing arrangements reinforced concrete frame structures, the exterior columns are made in pendent of the exterior walls and the walls are designed to span verti between the wall footing and the roof slab. This arrangement is desir and most economical because it eliminates the necessity of designing t column to act as a beam spanning between the foundation and the girder addition to providing restraint to the lateral motion of the frame.

7-15 PRELIMINARY DESIGN METHODS. The design of each element of a bui consists of two steps. The first step is the preliminary design of the element using an idealized straight line load-time curve and the desig charts presented in EM 1110-345-415. The second step is the numerical tegration check of the preliminary design using the calculated load-ti data. The following discussion deals with some of the details of the liminary design method.

Only one mass factor and one load factor may be used in any of t preliminary design methods. Therefore, average values of these factors must be obtained for all designs in the plastic range and also for ela designs in which there is a bilinear resistance function. In the case fixed-end beam designed to allow plastic deformation at midspan, the av age of the elasto-plastic and plastic mass and load factors should be u to obtain the mass and load factors for use in the preliminary design. elastic values of mass and load factors are not used in computing these average values since only a small percentage of the total deflection oc within the elastic range.

For a simple beam designed for plastic action, the average of the elastic and plastic values of mass and load factors should be used. In case of a fixed-end beam designed for plastic action at the support and elastic action at the centerline, the average of the elastic and

elasto-plastic mass and load factors is used in the preliminary design. In general, any reasonable method of obtaining a single set of mass and load factors will be satisfactory for use in the preliminary design.

7-16 NUMERICAL INTEGRATION ANALYSIS. The numerical integration analysis is a method of checking the preliminary design that takes into account the irregularity of the load-time curve, the variation in resistance function, and the changes in mass-load factors. In this manual each preliminary design is checked by the numerical integration method. In some cases, it may be necessary to use the numerical integration method more than once before a satisfactory design is obtained. This is particularly true where the actual load curve is of such a shape that a good approximation to it cannot be obtained by a straight line, and also in design of steel where the number of available beam sections is limited. An experienced designer may judge that a numerical integration check of some designs is unnecessary. A numerical integration analysis may be needed in some cases primarily to obtain the dynamic reactions of the element for use as the load on the supporting structure.

The load-time curves used in the numerical integration analysis are computed from either the direct-blast-pressure vs time curve or from the dynamic reactions of the supported elements. The dynamic reactions are computed by the use of the formulas of tables 6.1 to 6.6. Note that, in general, the formulas vary with the strain condition of the beam or slab. The dynamic reaction curve for an element designed to have some plastic action has a form indicated by line A in figure 7.6. Line B in figure 7.6 is the pseudostatic reaction (for a simple beam it equals one-half the applied load). The time t_1 indicates the first instance after the maximum reaction develops for which the dynamic reaction is less than the pseudostatic reaction.

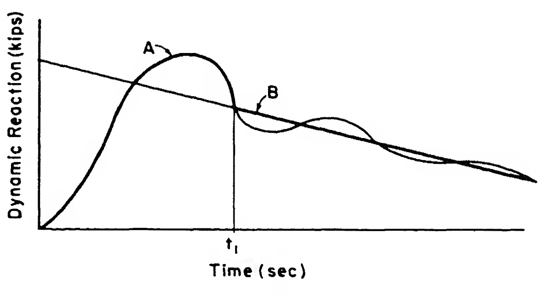


Figure 7.6. Simplification of dynamic reaction curve

If the dynamic load on the supporting structures is required for a period of time exceeding t_1, it is recommended that the load be represented

by line A before t_1 and by line B after t_1. This simplifies the load sh in a reasonable manner and reduces the number of tedious steps in the numerical integration.

7-17 SUMMARY OF DESIGN EXAMPLES. In this paragraph the results of all illustrative design examples which are contained in the remainder of thi manual are summarized to provide a ready reference for preliminary desi of similar elements and buildings. Examples are for incident overpressu and duration indicated in figure 7.8.

Table 7.1. Summary of Design Examples in EM 1110-345-417

Element	Span (ft)	Size	$\alpha\beta = \frac{x_m}{x_e}$	Maximum Deflection Range	C_R	T/T_n	D.L.F.	C_W	W_m (ft-kips)	E (ft-ki
R/C wall slab	17.5	11 in.	4.6	Plastic	0.70	1.07	----	0.365	7.42	7.6
R/C roof slab	6.67	3-3/4 in.	1.8	Plastic	1.3	18.3	----	0.0015	0.177	0.5
Steel purlin	18.0	16 WF 36	6.9	Plastic	1.07	10.3	----	0.008	15.4	22.0
Steel girder	20.0	36 WF 160	----	Elastic	----	0.69	1.38	------	-------	-----
Steel column	14.5	10 WF 77	10.1	Plastic	0.22	0.27	----	0.88	157.0	236.0
R/C wall slab	11.67	14 in.	----	Elastic	----	1.57	1.7	------	-------	-----
R/C roof slab	5.33	4-1/4 in.	----	Elastic	- --	37.0	2.0	------	-------	-----
Steel purlin	18.0	16 WF 40	----	Elasto-plastic	----	10.8	1.95	------	-------	-----
Steel girder	20.0	36 WF 150	----	Elastic	----	0.69	1.38	------	-------	-----
Steel column	8.67	12 WF 120	----	Elastic	----	0.593	1.3	------	-------	-----
R/C wall slab	15.83	10.5 in.	3.5	Plastic	0.77	1.24	----	0.31	5.95	6.4
R/C roof slab	18.0	7-3/4 in.	5.5	Plastic	1.0	5.7	----	0.028	4.55	4.5
R/C girder	20.0	20 in. x 45 in. T	----	Elastic	----	1.25	1.18	------	-------	-----
R/C column	13.0	12 in. x 20 in.	6.0	Plastic	0.324	0.338	----	0.83	92.1	114.0
R/C wall slab	14.75	12-1/2 in.	----	Elasto-plastic	1.26	1.67	----	0.118	1.76	1.7
R/C roof slab	18.0	9-1/2 in.	----	Elasto-plastic	----	6.6	1.92	------	-------	-----
R/C girder	16.0	20 in. x 42 in. T	----	Elastic	----	1.59	1.18	------	-------	-----
R/C column	11.83	18 in. x 28 in.	----	Elastic	----	0.48	1.15	------	-------	-----

NUMERICAL EXAMPLE, DESIGN OF A ONE-STORY STEEL FRAME BUILDING, PLASTIC DEFORMATION PERMITTED

7-18 GENERAL. This numerical example presents the design of one bay of windowless one-story, steel, rigid-frame building with plastic deformati permitted (fig. 7.7). The design overpressure (10 psi from an 18-KT weapon) is arbitrarily selected for illustrative purposes. In an actual case the design overpressure would be determined by evaluating a group c considerations including many nonstructural design considerations. The example includes only the designs of the major elements of the structure including the roof slab, purlins, wall slab, columns and girders of the frame, and the foundation.

One-way reinforced concrete slabs are used for the roof and walls.

The concrete compressive strength is specified at 3000 psi and intermediate grade reinforcing steel is used. In accordance with EM 1110-345-414, a uniform dynamic increase factor of 1.3 is used, giving the following strength properties for use in the design concrete

$$f'_c = 3000 \text{ psi}$$
$$f'_{dc} = 3900 \text{ psi}$$
$$E_c = 3(10)^6 \text{ psi}$$
$$n = 10$$

reinforcing steel

$$f_y = 40{,}000 \text{ psi}$$
$$f_{dy} = 52{,}000 \text{ psi}$$

The purlins, columns, and girders are wide flange structural shapes with welded connections. The strength properties are specified in EM 1110-345-414.

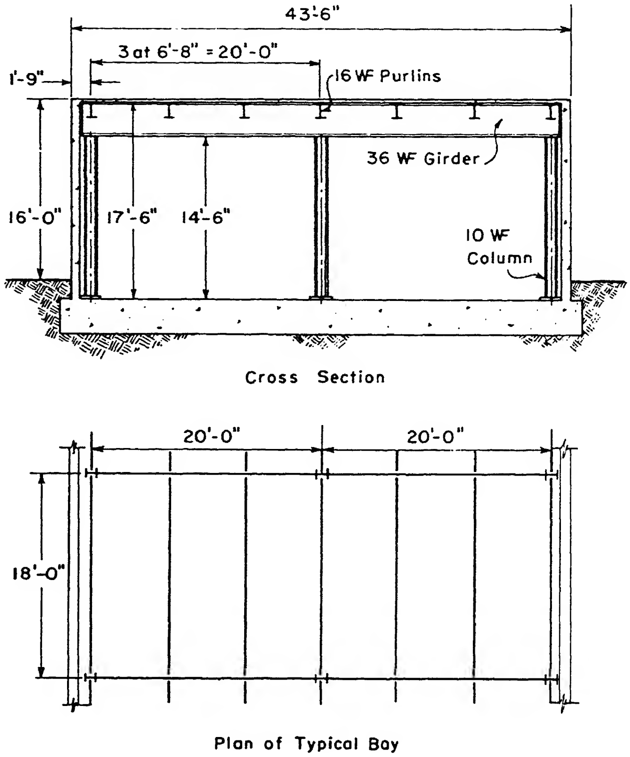


Figure 7.7. Plan and section of steel frame building

The structure is to be located upon a compact sand-gravel mixture exhibiting the following properties (par. 4-15).

Normal load-bearing capacity = 10 kips/sq ft

Ultimate load-bearing capacity = 30 kips/sq ft

Coefficient of friction (soil on soil) = 0.50

Coefficient of friction (concrete on soil) = 0.75

Unit weight of soil = 100 lb/ft^3

Normal component of passive pressure coefficient, $K_{P\phi} = 10$

Modulus of elasticity, E = 40,000 psi

It must be emphasized that the primary purposes of this example and those that follow are to illustrate the design techniques and philosophy presented in the previous manuals. Presentation on this example should not be considered a recommendation of the structural system for use in blast-resistant buildings.

7-19 <u>DESIGN PROCEDURE.</u> In general blast-resistant design procee the outside to the inside because the dynamic reactions of the ou elements are used for the loading on the supporting members. The the design procedure are as follows:

Step 1. Compute the pressure variations from which the des can be obtained. The following curves are needed in addition to roof overpressure curves which are introduced in paragraph 7-22:

(1) Incident overpressure vs time curve (fig. 7.8)
(2) Front face overpressure vs time curve (fig. 7.9)
(3) Rear face overpressure vs time curve (fig. 7.10)
(4) Net lateral overpressure vs time curve (fig. 7.11)
(5) Average roof overpressure vs time curve (fig. 7.12)

Step 2. Using the procedure of paragraph [illegible] for design w deformation and a triangular load-time curve idealized from the f overpressure-time curve make a preliminary design of a wall slab. the design using the numerical integration procedure of paragraph the computed front face overpressure-time curve. In this analysi namic reaction of the wall slab at the roof and foundation are ob later use.

Step 3. Design the roof slab by the same procedure used fo of the wall slab with a triangular load-time curve idealized from cident overpressure-time curve for the preliminary design, and use incident overpressure-time curve in the numerical integration to slab deflection. The dynamic reactions are obtained for use in t design.

Step 4. Base the purlin preliminary design on the same ide load-time curve used in the roof slab design. However, for the n integration analysis use the load obtained from the roof slab dyn action. From the numerical analysis of the purlin obtain the des for the girder.

Step 5. Make a preliminary design of the columns assuming to be infinitely rigid and neglecting the effect of axial load on column. A triangular load-time curve idealized from the net late pressure-time curve is used in the preliminary design.

Step 6. Design the frame girders using the procedure of paragraph 6-12 for elastic design. In this preliminary design the load-time curve is idealized from the variation of purlin dynamic reactions.

Step 7. Check the preliminary column design by determining the maximum lateral deflection of the frame considering the relative flexibility of the columns and girders and the effect of axial load on column resistance. Use the wall slab dynamic reactions at the roof line for the design lateral load on the frame.

Step 8. Design the foundation.

7-20 LOAD DETERMINATION. The computation of the various pressure-time curves is explained in detail in EM 1110-345-413. In this example the methods are illustrated by presenting the computations for one point on each of the curves. The dimensions of the structure used in the load computations are the outside dimensions of the building which are determined at this stage of the design by estimating the sizes of the slabs and girders (fig. 7.7).

a. Incident Overpressure vs Time Curve. Assumptions of the incident overpressure and time duration used in the illustrative examples in this manual are given in figure 7.8.

$$t_o = W^{1/3}\ (0.262) = 18^{1/3}\ (0.262) = 0.685 \text{ sec}$$

The incident overpressure-time curve (fig. 7.8) is obtained from figure 3.4b.

b. Front Face Overpressure vs Time Curve (Fig. 7.9).

c_{refl} = 1290 fps (fig. 3.21)

$$t_c = \frac{3h'}{c_{refl}} = \frac{3(16)}{1290} = 0.0372 \text{ sec}$$

P_{refl} = 25.3 psi (fig. 3.20)

q_o = 2.23 psi (fig. 3.23)

Overpressure = $P_s + 0.85q$

q is obtained using table 3.2. P_s is obtained using table 3.1.

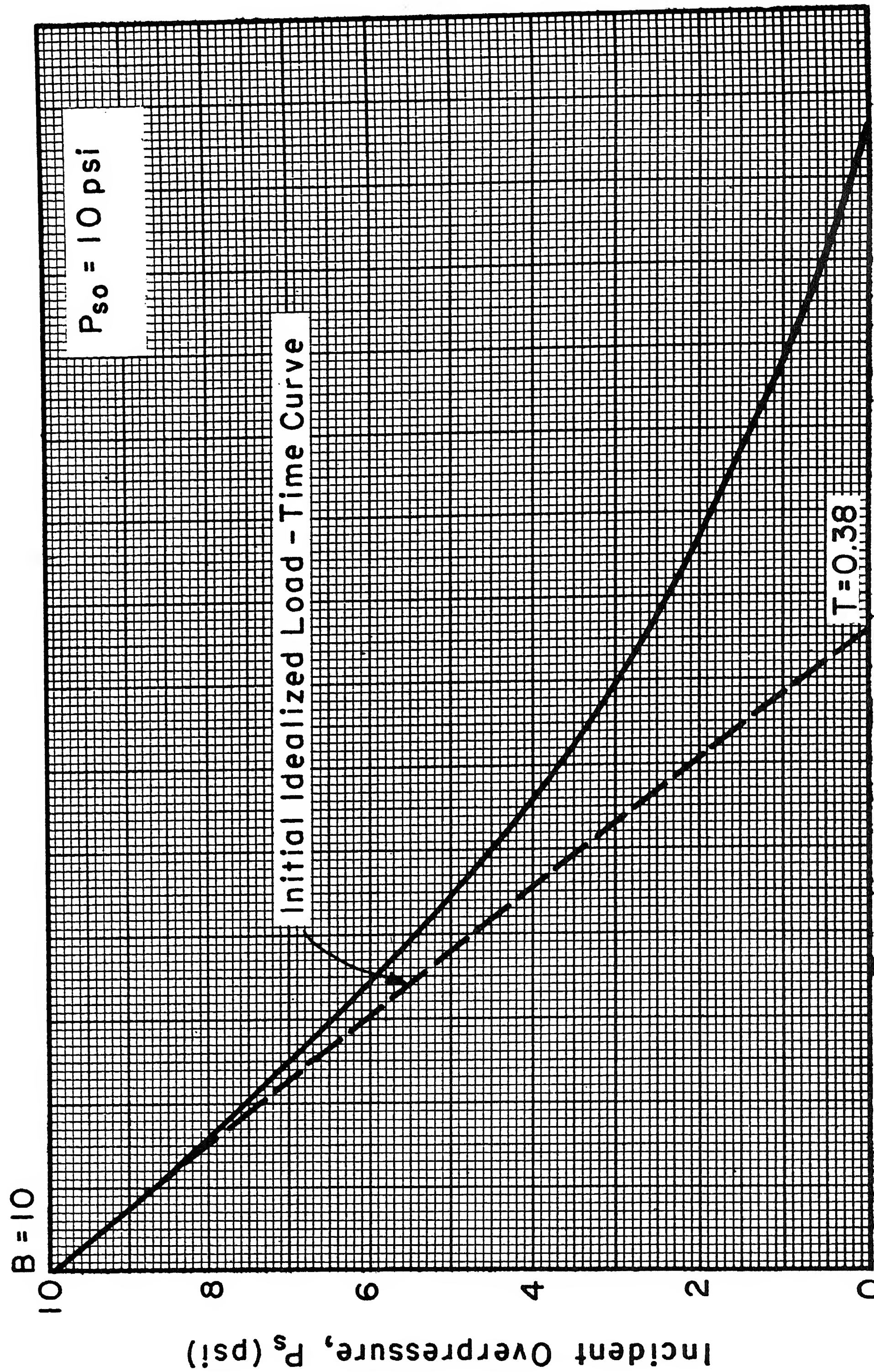
For example, for t = 0.100 sec, $t/t_o = 0.146$

q = 2.23(0.513) = 1.14 psi (table 3.2)

P_s = 10.0(0.738) = 7.38 psi (table 3.1)

$\bar{P}_{front}$ = 7.38 + 0.85(1.14) = 8.35 psi

Following the procedure of figure 3.25, the front face overpressure vs time curve may be drawn.


Pso = 10 psi
Initial Idealized Load - Time Curve
T = 0.38
B = 10
10
8
6
4
2
0
Incident Overpressure, Ps (psi)

Time, t (sec)

Figure 7.8. Incident overpressure vs time curve

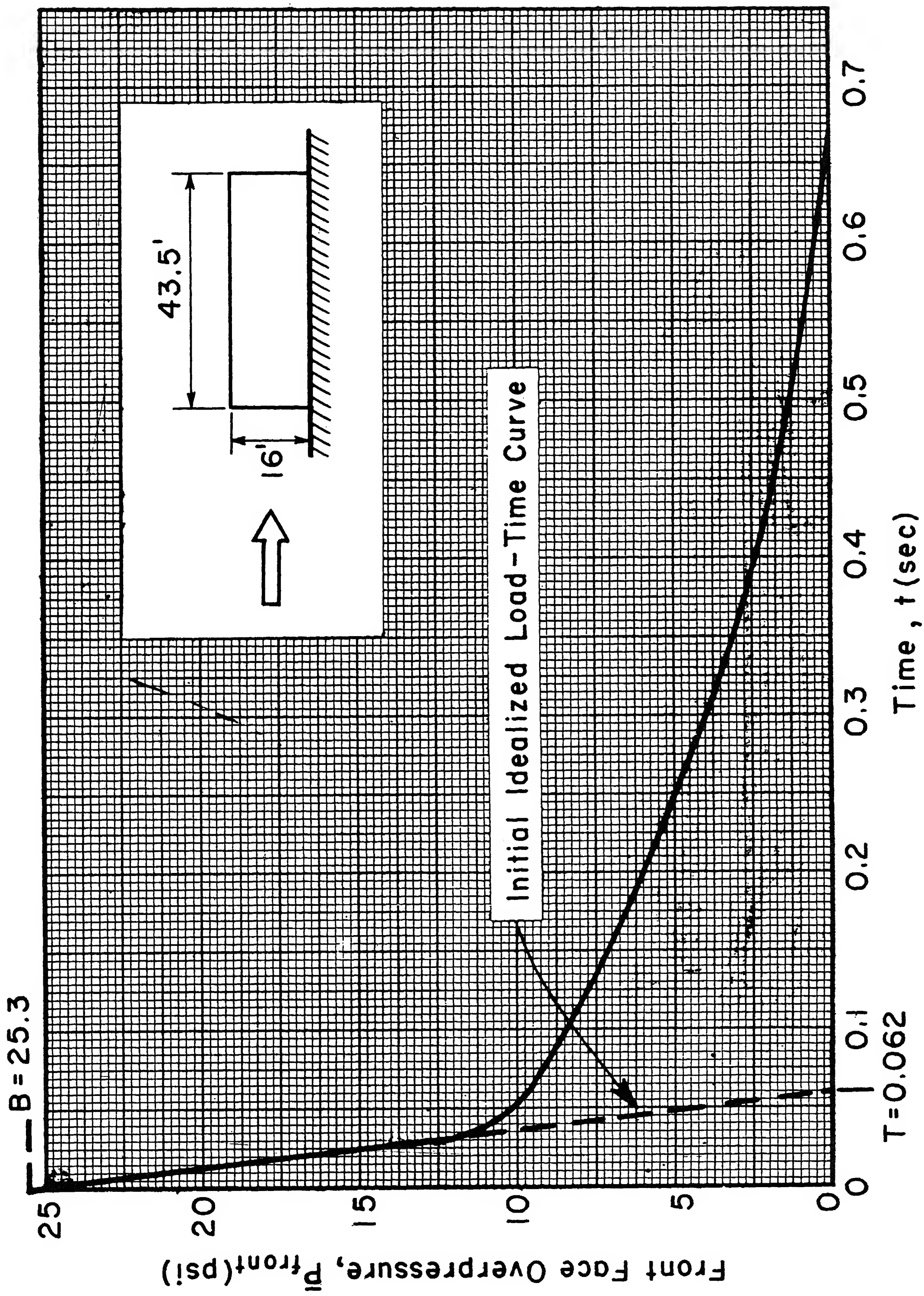
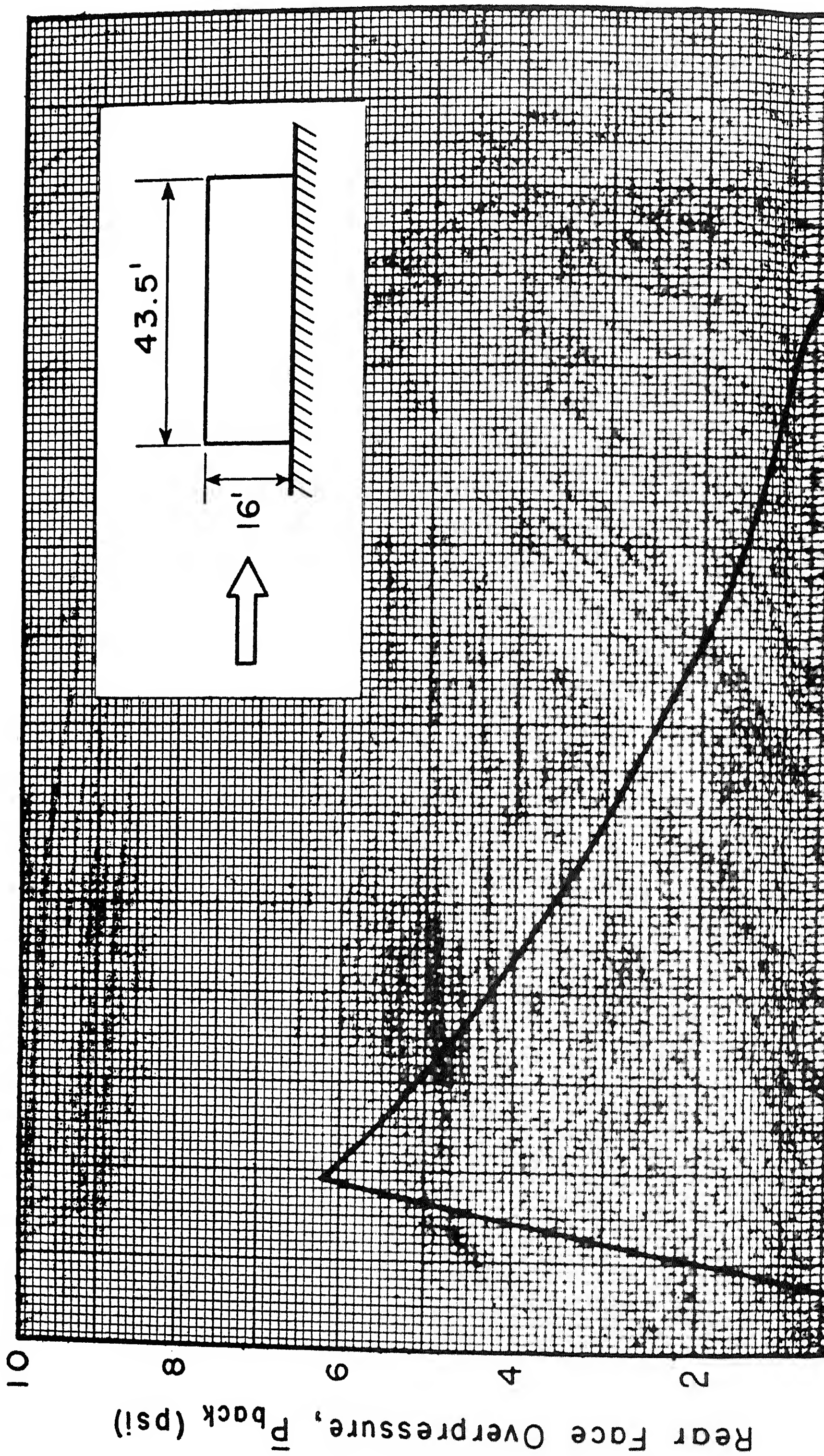


Figure 7.9. Front face overpressure vs time curve


43.5'
16'
10
8
6
4
2
Rear Face Overpressure, $\bar{P}_{back}$ (psi)

7-20b

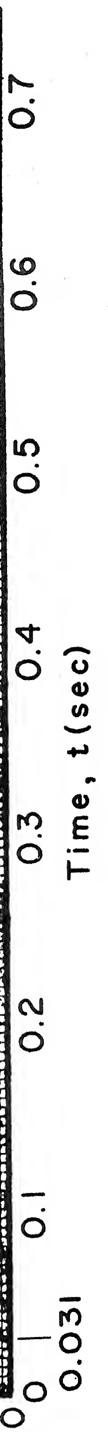


Figure 7.10. Rear face overpressure vs time curve

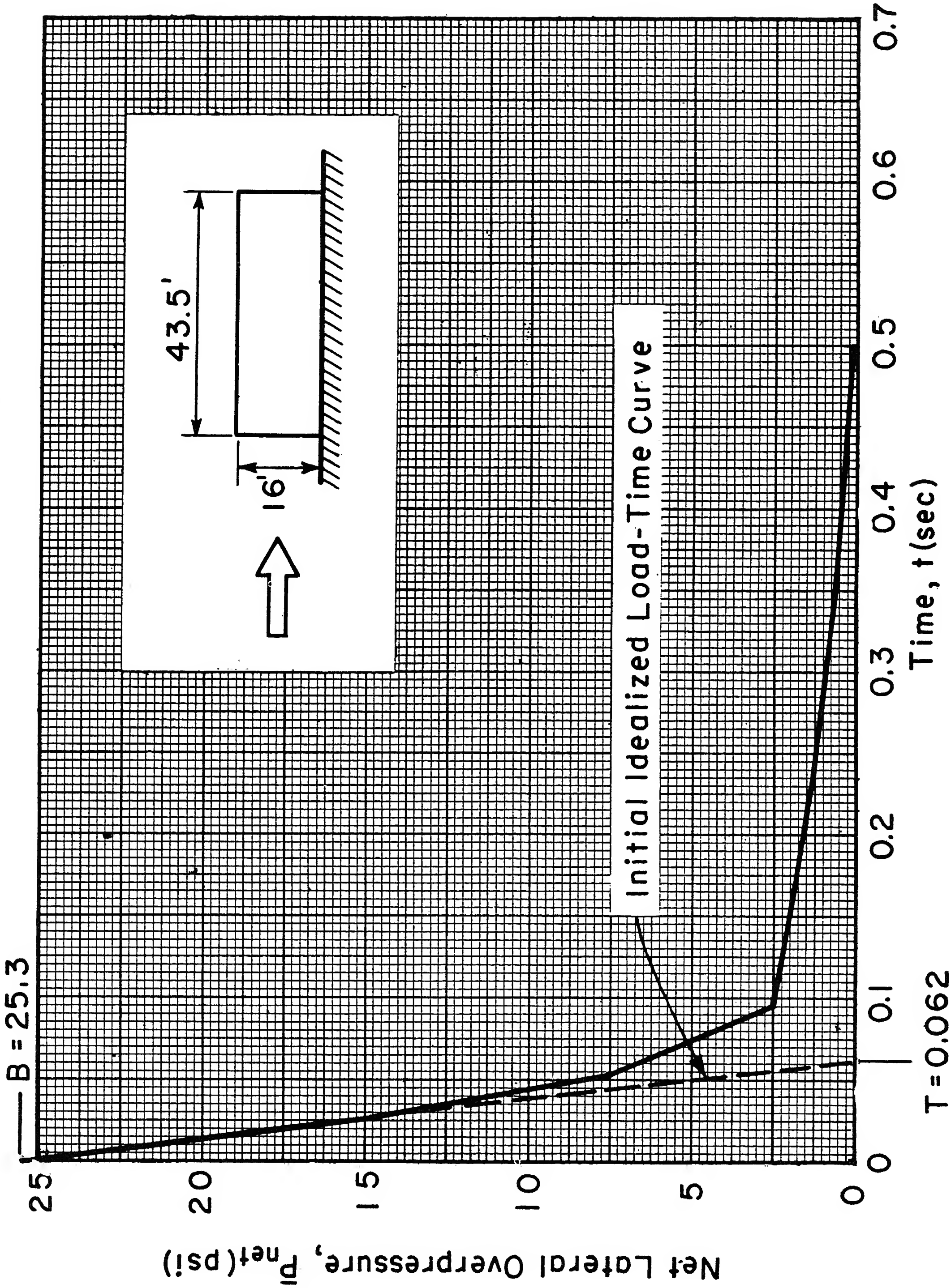
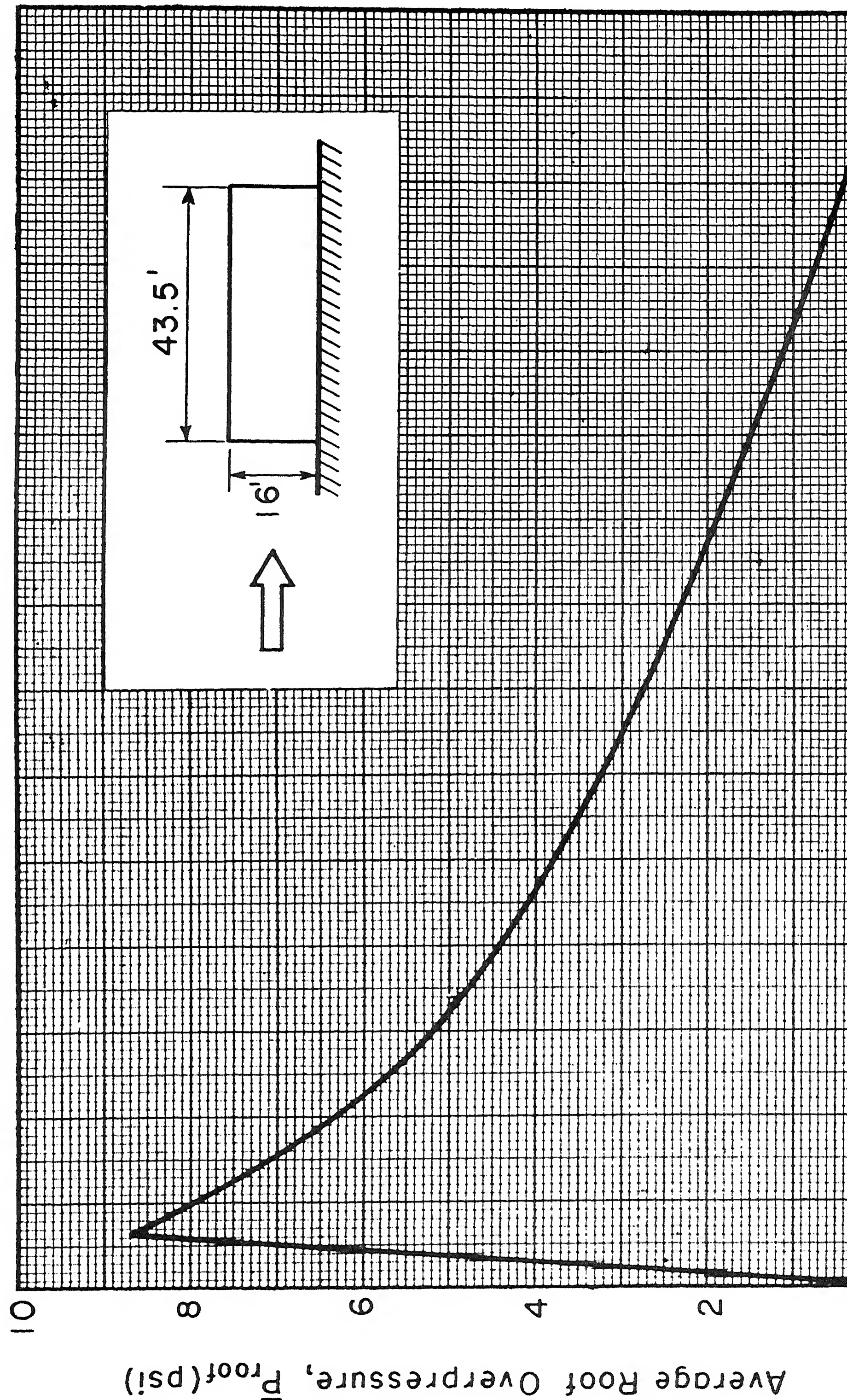


Figure 7.11. Net lateral overpressure vs time curve


43.5'
16'
10
8
6
4
2
Average Roof Overpressure, $\bar{P}_{roof}$ (psi)

c. Rear Face Overpressure vs Time Curve (Fig. 7.10).

Length of structure, L = 43.5 ft

U_o = 1403 fps (fig. 3.9), c_o = 1115 fps (par. 3-08a)

$$t_d = \frac{L}{U_o} = \frac{43.5}{1403} = 0.0310 \text{ sec}$$

$$t_b = \frac{4h'}{c_o} = \frac{4(16)}{1115} = 0.0573 \text{ sec}$$

At time $t_d + t_b$ = 0.088 sec, using table 3.1, P_{sb} = 10.0(0.842) = 8.42 psi

At time $t_d + t_b$, from figure 3.27b, $\overline{P}_{back}/P_{sb}$ = 0.735

Therefore, $\overline{P}_{back}$ = 0.735(8.42) = 6.20 psi

For times in excess of $t = t_d + t_b$, the ratio of $\overline{P}_{back}/P_s$ is as given in figure 3.27b.

d. Net Lateral Overpressure vs Time Curve (Fig. 7.11). At any time t, $\overline{P}_{net} = \overline{P}_{front} - \overline{P}_{back}$

e. Average Roof Overpressure (Fig. 7.12). For the blast propagation direction normal to the long side of the building, at time $t = L/U_o$ = 0.0310 sec

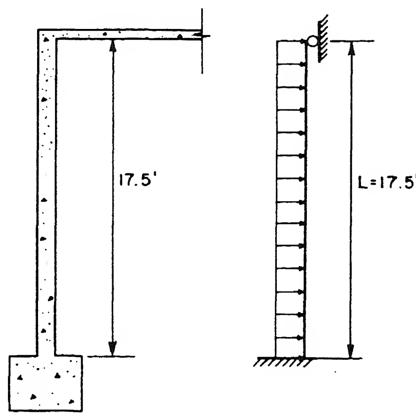
$$\frac{\overline{P}_{roof}}{P_s} = 0.9 + 0.1\left[\frac{1 - P_{so}}{14.7}\right]^2 = 0.91 \quad \text{(fig. 3.34)}$$

P_s = 10.0(0.957) = 9.57 psi (table 3.1)

Therefore, $\overline{P}_{roof}$ = 0.91(9.57) = 8.71 psi at t = 0.0310 sec

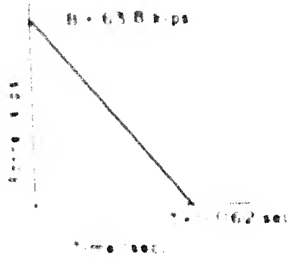
For times in excess of $t = L/U_o$, the ratio of $\overline{P}_{roof}/P_s$ is as given in figure 3.34.

7-21 DESIGN OF WALL SLAB. The wall is designed as a one-way reinforced concrete slab spanning from a fixed support at the foundation to a pinned support at the roof slab. The slab is permitted to deform into the plastic region by developing plastic hinges at the foundation and near midheight. The span length of the slab is equal to the clear height of the wall.



[illegible] preliminary plastic design procedure is described and [illegible] example in paragraph 6-11. Dead loads are not considered [illegible] wall slabs. The design calculations are made for [illegible] width of slab.

a. Design Loading. The design load as idealized from th[illegible] loading shown by figure 7.9 is defined by:



$$B = \frac{25.3(144)17.5}{1000} = 63.8 \text{ kips}$$

$$T = 0.062 \text{ sec}$$

$$H = \frac{BT}{2} = \frac{(63.8)0.062}{2} = 1.98 \text{ kip-sec (par.}$$

b. Dynamic Design Factors. (Refer[illegible] table 6.1.)

[illegible]:

K_L [illegible], $K_M = 0.45$, $K_{LM} = 0.$[illegible]

R_m [illegible], $k_1 = \frac{185EI}{L^3}$

V [illegible], $V = 0.43R + 0.19P$

[illegible]:

K_L [illegible], K_M [illegible] 0.50, $K_{LM} = 0.$[illegible]

R_m [illegible] $(M_{[illegible]} + [illegible] M_{Pm})$, k_{e1} [illegible] $\frac{384EI}{5L^3}$

V [illegible]

[illegible]:

K_L [illegible], K_M [illegible], $K_{LM} = 0$[illegible]

R_m [illegible] $(M_{[illegible]} + [illegible] M_{Pm})$

V [illegible]

[illegible]:

K_L [illegible]

K_M [illegible]

R_m [illegible] $(M_{Ps} + [illegible] M_{Pm})$

k_E [illegible] $\frac{[illegible] EI}{L^3}$

c. First Trial - Actual Properties.

Let $M_{Ps} = M_{Pm} = M_P$

Assume $p = 0.015$ (par. 4-10)

Let $\alpha\beta = 5$ (par. 6-26)

Assume $C_R = 0.7$ (experience)

$R_m = C_R B = 0.7(63.8) = 44.7$ kips

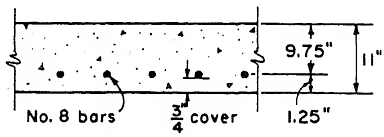
$$M_P = pf_{dy}bd^2\left(1 - \frac{pf_{dy}}{1.7f'_{dc}}\right) \quad \text{(eq 4.16)}$$

$$= 0.015(52)\ (1)d^2\left[1 - \frac{0.015(52)}{1.7(3.9)}\right] = 0.688d^2 \text{ kip-ft (d in inches)}$$

$$R_m = \frac{12M_P}{L} = \frac{(12)0.688d^2}{17.5} = 44.7,\ \therefore\ d = 9.73 \text{ in.}$$

Try $h = 11$ in., $d = 9.75$ in., $p = 0.015$

$M_P = 0.688(9.75)^2 = 65.4$ kip-ft



$$R_m = \frac{12M_P}{L} = \frac{12(65.4)}{17.5} = 44.8 \text{ kips}$$

$I_g = bh^3/12 = (11)^3 = 1331$ in.^{4}

$$I_t = bd^3\left[\frac{k^3}{3} + p(1-k)^2\right]$$

$$= 12(d)^3\left[\frac{(0.42)^3}{3} + 0.015(1 - 0.42)^2\right]$$

$$= 0.905d^3 = 0.905(9.75)^3 = 839 \text{ in.}^4$$

$I_a = 0.5(I_g + I_t) = 0.5(1331 + 839) = 1085$ in.^{4}

$$k_E = \frac{160EI}{L^3} = \frac{(160)3(10)^3 1085}{144(17.5)^3} = 675 \text{ kips/ft}$$

$$y_E = \frac{R_m}{k_E} = \frac{44.8}{675} = 0.0664 \text{ ft}$$

$y_m = \alpha\beta\ y_E = 5(0.0664) = 0.3320$ ft (par. 6-26)

$$\text{Weight} = \frac{11(150)17.5}{(12)1000} = 2.406 \text{ kips}$$

$$\text{Mass}\quad m = \frac{2.406}{32.2} = 0.0747\ \frac{\text{kip-sec}^2}{\text{ft}}$$

d. First Trial - Equivalent System Properties.

$R_{me} = K_L R_m = 0.57(44.8) = 25.5$ kips (eq 6.12)

[illegible] kip-sec (eq 6.2)

[illegible] kip-sec^2/ft (eq 6.2)

[illegible] ft-kips (eq 6.10)

[illegible] = 0.058 sec (eq 6.1

[illegible] Capacity.

[illegible] (eqs [illegible], 6.16)

[illegible] satisfactory since it ag
[illegible] (fig. 7.9) (see par. 5-

[illegible] ft-kips (eq 6.17)
[illegible] [[illegible] - 0.5(0.0664)]

[illegible] are satisfactory as a p

[illegible] Stress. It is now necessa
[illegible] the reinforcing steel for th
[illegible] At the fixed end c
[illegible] requirement results in a
[illegible] in. than at midspa
[illegible] achieve approximately the s
[illegible] sections several valu
[illegible] to obtain the valu

$$\frac{M_{Pm} + 2M_{Ps}}{3} = M_P = 65.4$$ [illegible]

[illegible] (4.16) simplifies

<u>At the fixed end</u>

$$\text{Estimated } V_{max} = 0.5R_m = 0.5(44.8) = 22.4 \text{ kips}$$

$$\text{Allowable } u = 0.15f'_c = 0.15(3000) = 450 \text{ psi}$$

$$\Sigma o = \frac{V}{ujd} = \frac{8(22,400)}{7(450)8.5} = 6.7 \text{ in.}$$

$$A_s = pbd = 0.016(12)(8.5) = 1.63 \text{ in.}^2$$

Try #8 at 5 in., $A_s = 1.9$ in.^{2}, $\Sigma o = 7.5$ in.

$$p = \frac{1.9}{12(8.5)} = 0.0186$$

<u>At the pinned end</u>

$$\text{Estimated } V_{max} = (1/3)R_m = 1/3(44.8) = 15.0 \text{ kips}$$

$$\Sigma o = \frac{V}{ujd} = \frac{8(15,000)}{7(450)9.75} = 3.9 \text{ in.}$$

$$A_s = pbd = 0.016(12)(9.75) = 1.87 \text{ in.}^2$$

Try #8 at 5 in., $A_s = 1.9$ in.^{2}, $\Sigma o = 7.5$ in.

$$p = \frac{1.9}{12(9.75)} = 0.0162$$

g. <u>Determination of Maximum Deflection and Dynamic Reactions by Numerical Integration.</u>

$$M_{Pm} = pf_{dy}bd^2\left[1 - \frac{pf_{dy}}{1.7f'_{dc}}\right] = 0.0162(52)(1)(9.75)^2\left[1 - \frac{(0.0162)52}{1.7(3.9)}\right]$$

$$= 70.0 \text{ kip-ft (eq 4.16)}$$

$$M_{Ps} = 0.0186(52)(1)(8.5)^2\left[1 - \frac{0.0186(52)}{1.7(3.9)}\right] = 59.5 \text{ kip-ft}$$

$$I_g = bh^3/12 = (11)^3 = 1331 \text{ in.}^4$$

$$I_t = bd^3\left[k^3/3 + np(1-k)^2\right] = 12(9.75)^3\left[\frac{0.43^3}{3} + 0.162(1-0.43)^2\right]$$

$$= 880 \text{ in.}^4$$

$$I_a = 0.5(I_g + I_t) = 0.5(1331 + 880) = 1105 \text{ in.}^4$$

$$\text{Weight} = \frac{11(150)17.5}{12(1000)} = 2.406 \text{ kips}$$

$$\text{Mass } m = \frac{2.406}{32.2} = 0.0747 \text{ kip-sec}^2/\text{ft}$$

Elastic range:

$$R_{1m} = \frac{8M_{Ps}}{L} = \frac{8(59.5)}{17.5} = 27.2 \text{ kips}$$

$$k_1 = \frac{185EI}{L^3} = \frac{(185)3(10)^3 1105}{144(17.5)^3} = 793 \text{ kips/ft}$$

$$y_e = \frac{R_{1m}}{k_1} = \frac{27.2}{793} = 0.0344 \text{ ft}$$

Elasto-plastic range:

$$R_m = 4\left(\frac{M_{Ps} + 2M_{Pm}}{L}\right) = \frac{4[59.5 + 2(70)]}{17.5} = 45.6 \text{ kips}$$

$$k_{ep} = \frac{384EI}{5L^3} = \frac{384}{5(185)} k_1 = 329 \text{ kips/ft}$$

$$y_{ep} = y_e + \frac{R_m - R_{1m}}{k_{ep}} = 0.0344 + \frac{45.6 - 27.2}{329} = 0.090 \text{ ft}$$

Plastic range:

$$R_m = 45.6 \text{ kips}$$

Since $M_{Pm} \neq M_{Ps}$ the formula for k_E in table 6.1 is no
obtain a value for y_E and k_E, an "effective resistance" lin
figure 7.13 so that the ar
to y_E is equal to the area
culated elasto-plastic res
The required value of y_E =

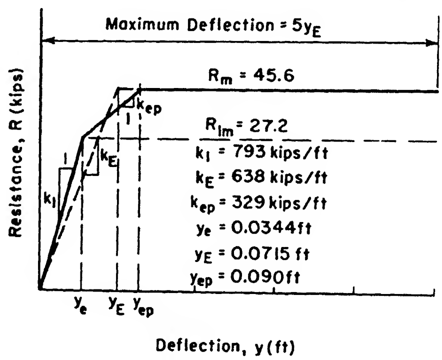


Figure 7.13. Resistance function for 11-in. wall slab spanning 17.5 ft fixed at base and pinned at top

$$k_E = \frac{R_m}{y_E} = \frac{45.6}{0.0715} =$$

$$y_m = \alpha\beta y_E = 5(0.071$$

$$T_n = 2\pi\sqrt{\frac{K_{LM}m}{k_E}} = 6$$

$$= 0.06 \text{ sec}$$

The basic equation
integration in table 7.2 is $y_{n+1} = \ddot{y}_n(\Delta t)^2 + 2y_n - y_{n-1}$
where

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(\Delta t)^2}{K_{LM}(m)}$$

Table 7.2. Determination of Maximum Deflection and Dynamic Reactions for Front Wall Slab

t (sec)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)	V_{1n} (kips)	V_{2n} (kips)
0	63.8	0	31.9	0.01369	0	7.7	12.1
.005	58.7	10.8	47.9	0.02055	0.01369	9.8	15.8
.010	53.5	31.6	21.9	0.00939	0.04793	18.2	18.2
.015	48.4	45.6	2.8	0.00142	0.09156	23.1	23.1
.020	43.2	45.6	-2.4	-0.00122	0.13661	22.5	22.5
.025	38.1	45.6	-7.5	-0.00380	0.18044	21.9	21.9
.030	32.9	45.6	-12.7	-0.00644	0.22047	21.3	21.3
.035	29.0	45.6	-16.6	-0.00842	0.25406	20.8	20.8
.040	27.3	45.6	-18.3	-0.00928	0.27923	20.6	20.6
.045	26.3	45.6	-19.3	-0.00979	0.29512	20.4	20.4
.050	25.6	45.6	-20.0	-0.01014	0.30122*	20.4	20.4
.055	24.9	42.4	-17.5	-0.00751	0.29718	14.0	22.9
.060	24.4	33.2	-8.8	-0.00378	0.28563	11.5	18.9
.065	23.9	21.1			0.27030	8.4	13.6

* $(y_n)_{max}$ = 0.30 ft.

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)25(10^{-6})}{0.78(0.0747)} = 4.29(10^{-4})(P_n - R_n) \text{ ft, elastic range}$$

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)25(10^{-6})}{0.78(0.0747)} = 4.29(10^{-4})(P_n - R_n) \text{ ft, elasto-plastic range}$$

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)25(10^{-6})}{0.66(0.0747)} = 5.07(10^{-4})(P_n - R_n) \text{ ft, plastic range}$$

The time interval Δt = 0.005 sec is approximately $T_n/10$ = 0.006 (par. 5-08).

The dynamic reaction equations are listed in paragraph 7-21b. The P_n values for the second column are obtained from figure 7.9, multiplying by 144(17.5)/1000 = 2.52.

The maximum deflection, $(y_n)_{max}$, computed in table 7.2, is 0.30 ft which is less than the allowable y_m of 0.35 ft.

y_E = 0.068 ft

$$\alpha\beta = \frac{(y_n)_{max}}{y_E} = \frac{0.30}{0.0715} = 4.2 < 5.0; \text{ OK}$$

... trength. For bottom of wall (fixed end

... (table 7.…)

... reinforcement (eq 4.24a)

… = 0.04(3000) + 5000(0.0162) = 120 + 81 =

… = …58 psi

… required for 253 - 201 = 57 psi. Contribution

… shear stress = rf_y.

… in., use s = 7.5 in.

… (idealized slab):

… (table …)

v_y = 201 psi

$$v = \frac{8V}{7bd} = \frac{8(23,100)}{7(12)9.75} = 22\ldots$$

Shear reinforcement required f…

…6 - 201 = 25 psi

r = 25/40,000 = 0.0006

Try 1 #2, A_s = 0.05 in.^{2}

$$r = \frac{A_s}{bs} = \frac{0.05}{10s} = 0.0006,$$

∴ s = 8.3 in.,

use s = 8 in.

Bond:

$$u = \frac{8V}{7\Sigma od} = \frac{8(23,100)}{7(7.5)(8.5)} = \ldots$$

Allowable u = 0.15 f'_c

= 0.15(3000) = 450 psi

… psi; OK

… Summary. 11-in. sla…

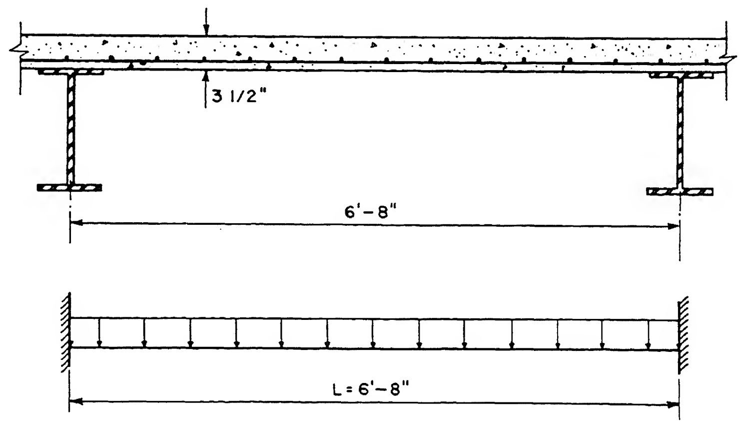
Shear reinforcement:

Bottom of wall #3 at 7-1/2 in.

Top of wall #3 at 8 in.

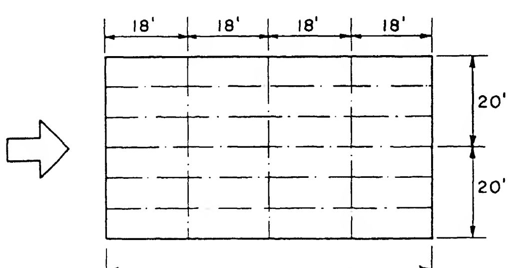
7-22 DESIGN OF ROOF SLAB. The roof slab is designed as a one-way reinforced concrete slab spanning continuously over purlins located at the third-points of the supporting girder.

As stated previously, the general arrangement of the members in this example is not selected as the result of economic studies. This example is intended primarily to illustrate design technique.



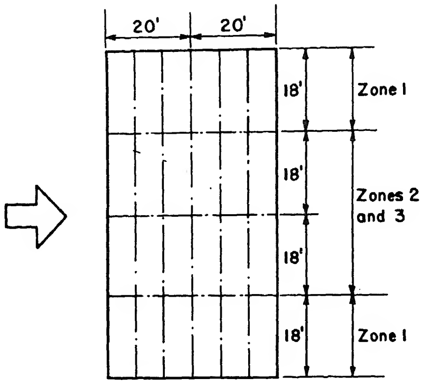
The slab is permitted to deform into the plastic region by developing plastic hinges at both supports and midspan. In the design procedures of this manual only single-span elements can be handled, therefore in using the preliminary plastic design procedure of paragraph 6-11 a one-foot width of slab is considered to be a fixed-end beam spanning 6 ft 8 in., the purlin spacing.

a. Loading. The critical slab loading is the incident overpressure vs time curve (fig. 7.8). This loading results from the blast wave moving parallel to the long axis of the building. Since the slab is framed perpendicular to the direction of the blast wave the load may be considered to be uniformly distributed along each slab span. The individual one-foot slab elements along the purlin reach their maximum deflections at different times; however, they provide little restraint to adjacent elements. This effect is neglected in this example.



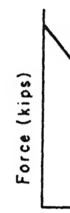
Zone I - Incident overpressure at all points of roof.

For the blast wave moving perpendicular to the long axis



building the design loads on th
reduced from the incident overp
loads for two reasons: (1) the
to build up load on the slab is
(2) the Zone 3 overpressures ar
the incident overpressure (EM 1
The design load as ideali
computed loading shown by figur
defined by

$$B = 10 \text{ psi} = \frac{10(144)6.67}{1000} = 9.6 \text{ kips}$$



$$T = 0.38 \text{ sec}$$

$$H = \frac{BT}{2} = \frac{(9.6)0.38}{2} = 1.83 \text{ kip-sec (par. 6-11)}$$

b. Dynamic Design Factors. (Refer to table 6.1.)

Elastic range:

$K_L = 0.53,$ $K_M = 0.41,$ $K_{LM} = 0.$

$R_{1m} = 12 \dfrac{M_{Ps}}{L},$ $k_1 = \dfrac{384EI}{L^3}$

$V = 0.36R + 0.14P$

Elasto-plastic range:

$K_L = 0.64,$ $K_M = 0.50,$ $K_{LM} = 0.$

$R_m = \dfrac{8}{L}(M_{Ps} + M_{Pm}),$ $k_{ep} = \dfrac{384EI}{5L^3}$

$V = 0.39R + 0.11P$

Plastic range:

$K_L = 0.50,$ $K_M = 0.33,$ $K_{LM} = 0.$

$R_m = \dfrac{8}{L}(M_{Ps} + M_{Pm})$

$V = 0.38R_m + 0.12P$

Average values:

$K_L = 0.5(0.64 + 0.50) = 0.57,$ $K_{LM} = 0.5(0.78 + 0.77) =$

$K_M = 0.5(0.50 + 0.33) = 0.42$

$$R_m = \frac{8}{L}(M_{Ps} + M_{Pm})$$

$$k_E = \frac{307EI}{L^3}$$

c. <u>First Trial - Actual Properties.</u>

Let $M_{Ps} = M_{Pm} = M_P$

Assume $p = 0.015$ (par. 4-10)

Let $\alpha\beta = 5$ (par. 6-26)

Assume $C_R = 1.0$ (experience)

$R_m = C_R B = 1.0(9.6) = 9.6$ kips

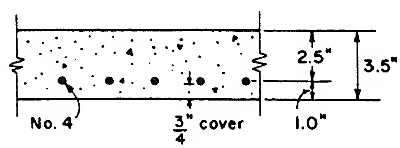
$$M_P = pf_{dy}bd^2\left(1 - \frac{pf_{dy}}{1.7f'_{dc}}\right) \text{ (eq 4.16)}$$

$$= 0.015(52)(1)d^2\left[1 - \frac{0.015(52)}{1.7(3.9)}\right] = 0.688d^2 \text{ kip-ft (d in inches)}$$

$$R_m = \frac{16M_P}{L} = \frac{(16)0.688d^2}{6.67} = 9.6, \therefore d = 2.4 \text{ in.}$$

Try $h = 3.5$ in., $d = 2.5$ in., $p = 0.015$, $np = 0.15$

$M_P = 0.688(2.5)^2 = 4.3$ kip-ft



$$R_m = \frac{16M_P}{L} = \frac{16(4.3)}{6.67} = 10.3 \text{ kips}$$

$I_g = bh^3/12 = (3.5)^3 = 42.9$ in.^{4}

$$I_t = bd^3\left[\frac{k^3}{3} + np(1 - k)^2\right]$$

$$= 12(d)^3\left[\frac{(0.42)^3}{3} + 0.15(1 - 0.42)^2\right] = 0.905d^3 = 0.905(2.5)^3$$

$$= 14.1 \text{ in.}^4$$

$I_a = 0.5(I_g + I_t) = 0.5(14.1 + 42.9) = 28.5$ in.^{4}

$$k_E = \frac{307EI}{L^3} = \frac{(307)3(10)^3 28.5}{(6.67)^3 144} = 615 \text{ kips/ft}$$

$$y_E = \frac{R_m}{k_E} = \frac{10.3}{615} = 0.0168 \text{ ft}$$

$y_m = \alpha\beta\, y_e = 5(0.0168) = 0.084$ ft (par. 6-26)

The roofing weight is 6 psf.

$$\text{Weight} = \left[\frac{3.5(150)}{(12)} + 6.0\right]\frac{6.67}{1000} = 0.332 \text{ kips}$$

$$\text{Mass } m = \frac{0.332}{32.2} = 0.0103 \text{ kip-sec}^2/\text{ft}$$

d. <u>First Trial - Equivalent System Properties.</u>

$$R_{me} = K_L R_m = 0.57(10.3) = 5.87 \text{ kips (eq 6.12)}$$

$$H_e = K_L H = 0.57(1.83) = 1.04 \text{ kip-sec (eq 6.2)}$$

$$m_e = K_M m = 0.42(0.0103) = 0.00432 \text{ kip-sec}^2/\text{ft (eq 6.2)}$$

$$W_P = \frac{(H_e)^2}{2m_e} = \frac{(1.04)^2}{2(0.00432)} = 125 \text{ ft-kips (eq 6.10)}$$

$$T_n = 2\pi\sqrt{K_{LM}\frac{m}{k_E}} = 6.28\sqrt{0.77(0.0103)/615} = 0.0226 \text{ sec}$$

e. <u>Work Done vs Energy Absorption Capacity.</u>

$$C_T = T/T_n = 0.38/0.0226 = 16.9$$

$$C_R = R_m/B = 10.3/9.6 = 1.075 \text{ (eqs 6.15, 6.16)}$$

$$t_m/T = 0.09 \text{ (fig. 5.29)}$$

$$t_m = (0.09)0.38 = 0.034 \text{ sec}$$

Idealized load-time curve is satisfactory at $t = 0.034$

$$C_W = 0.005 \text{ (fig. 5.27)}$$

$$W_m = C_W W_P = 0.005\ (1.25) = 0.625 \text{ ft-kips (eq 6.17)}$$

$$E = R_{me}(y_m - 0.5y_E) = 5.87\ [0.084 - 0.5(0.0168)]$$

$$= 0.445 \text{ ft-kips (eq 6.18)}$$

$E < W$, $\therefore$ the selected proportions are unsatisfactory as
nary design.

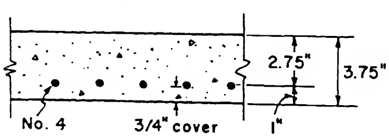
f. <u>Second Trial - Actual Properties.</u>

$$R_m = K_L\frac{0.5(W_m + E)}{(y_m - 0.5y_E)} = \frac{0.5(0.625 + 0.445)}{(0.57)\ [0.084 - 0.5(0.0168)]}$$

$$= 12.3 \text{ kips (eq 6.19)}$$

$$R_m = \frac{16M_P}{L} = \frac{(16)0.688d^2}{6.67} = 12.3, \quad \therefore\ d = 2.7 \text{ in.}$$

Try $h = 3\text{-}3/4$ in., $d = 2\text{-}3/4$ in., $p = 0.015$



$$M_P = 0.688d^2 = 0.688(2.75)^2 = 5.2$$

$$R_m = \frac{16M_P}{L} = \frac{16(5.2)}{6.67} = 12.5 \text{ kips}$$

$$I_g = \frac{bh^3}{12} = (3.75)^3 = 53.0 \text{ in.}^4$$

$$I_t = 0.905d^3 = 0.905(2.75)^3 = 18.8 \text{ in.}^4 \quad (k = 0.42)$$

$$I_a = 0.5(I_g + I_t) = 0.5(53.0 + 18.8) = 35.9 \text{ in.}^4$$

$$k_E = \frac{307EI}{L^3} = \frac{(307)3(10)^3 35.9}{(6.67)^3(144)} = 775 \text{ kips/ft}$$

$$y_E = \frac{R_m}{k_E} = \frac{12.5}{775} = 0.0161 \text{ ft}$$

4) $$y_m = \alpha\beta\, y_E = 5(0.0161) = 0.0805 \text{ ft (par. 6-26)}$$

$$\text{Weight} = \left[\frac{3.75(150) + 6.0}{(12)}\right]\frac{6.67}{1000} = 0.352 \text{ kips}$$

$$\text{Mass } m = \frac{0.352}{32.2} = 0.0109 \text{ kip-sec}^2\text{/ft}$$

g. <u>Second Trial - Equivalent System Properties.</u>

$$R_{me} = K_L R_m = 0.57(12.5) = 7.13 \text{ kips (eq 6.12)}$$

$$H_e = K_L H = 0.57(1.83) = 1.04 \text{ kip-sec (eq 6.2)}$$

$$m_e = K_M m = 0.42(0.0109) = 0.00459 \text{ kip-sec}^2\text{/ft (eq 6.2)}$$

$$W_P = \frac{(H_e)^2}{2m_e} = \frac{(1.04)^2}{2(0.00459)} = 118 \text{ ft-kips (eq 6.10)}$$

$$T_n = 2\pi\sqrt{K_{LM}m/k_E} = 6.28\sqrt{0.77(0.0109)/775} = 0.0207 \text{ sec}$$

imi-

h. <u>Work Done vs Energy Absorption Capacity.</u>

$$C_T = T/T_n = 0.38/0.0207 = 18.3$$

$$C_R = R_m/B = 12.5/9.6 = 1.3 \text{ (eqs 6.15, 6.16)}$$

$$t_m/T = 0.04 \text{ (fig. 5.29)}$$

$$t_m = (0.04)0.38 = 0.015 \text{ sec}$$

$$C_W = 0.0015 \text{ (fig. 5.27)}$$

$$W_m = C_W W_P = 0.0015(118) = 0.177 \text{ ft-kips (eq 6.17)}$$

$$E = R_{me}(y_m - 0.5y_E) = 7.13\,[0.0805 - 0.5(0.0161)]$$
$$= 0.516 \text{ ft-kips (eq 6.18)}$$

$$E \gg W$$

Although the difference between E and W is great no other trial

is justified since slab thickness was increased by only 1/4 i
the selected proportions are satisfactory as a preliminary de

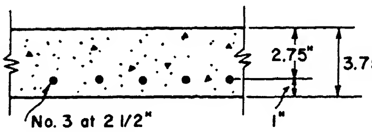
i. Preliminary Design for Bond Stress.

Estimated $V_{max} = 0.5R_m = 0.5(12.5) = 6.25$ kips

Allowable $u = 0.15f'_c = 0.15(3000) = 450$ psi (par. 4-09)

$$\Sigma o = \frac{V}{ujd} = \frac{8(6250)}{450(7)2.75} = 5.77 \text{ in.}$$

Try #3 at 2-1/2 in., $A_s = 0.53$ in.^{2}, $\Sigma o = 5.7$ in.



$$p = A_s/bd = \frac{0.53}{12(2.75)} = 0.16,$$

$$np = 10(0.016) = 0.16$$

j. Determination of Maximum Deflection and Dynamic Rea
Numerical Integration.

$$M_P = pf_{dy}bd^2 \left[1 - \frac{pf_{dy}}{1.7f'_{dc}}\right]$$

$$= 0.016(52)(1)(2.75)^2 \left[1 - \frac{(0.016)52}{1.7(3.9)}\right] = 5.5 \text{ kip-ft (}$$

$$I_g = bh^3/12 = (3.75)^3 = 53.0 \text{ in.}^4$$

$$k = \sqrt{n^2p^2 + 2np} - np = 0.428$$

$$I_t = bd^3 \left[k^3/3 + np(1 - k)^2\right]$$

$$= 12(2.75)^3 \left[\frac{0.428^3}{3} + 0.16(1 - 0.428)^2\right] = 0.945(2.75$$

$$I_a = 0.5(I_g + I_t) = 0.5(53.0 + 19.6) = 36.3 \text{ in.}^4$$

$$\text{Weight} = \left[\frac{3.75(150)}{12} + 6.0\right]\frac{6.67}{1000} = 0.352 \text{ kips}$$

$$\text{Mass } m = \frac{0.352}{32.2} = 0.0109 \text{ kip-sec}^2\text{/ft}$$

Elastic range:

$$R_{1m} = \frac{12M_P}{L} - \text{weight} = \frac{12(5.5)}{6.67} - 0.352 = 9.5 \text{ kips}$$

$$k_1 = \frac{384EI}{L^3} = \frac{(384)3(10)^3 36.3}{(6.67)^3 144} = 980 \text{ kips/ft}$$

$$y_e = \frac{R_{1m}}{k_1} = \frac{9.5}{980} = 0.0097 \text{ ft}$$

Elasto-plastic range:

$$R_m = \frac{16M_P}{L} - \text{weight} = \frac{16(5.5)}{6.67} - 0.352 = 12.9 \text{ kips}$$

$$k_{ep} = \frac{384EI}{5L^3} = \frac{1}{5}\, k_1 = 196 \text{ kips/ft}$$

$$y_{ep} = y_e + \frac{R_m - R_{1m}}{k_{ep}} = 0.0097 + \frac{12.9 - 9.5}{196} = 0.027 \text{ ft}$$

Plastic range:

$$R_m = \frac{16M_P}{L} - \text{weight} = 12.9 \text{ kips}$$

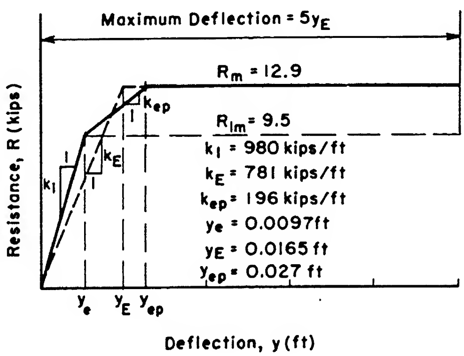


Figure 7.14. Resistance function for 3-3/4-in. slab spanning 6.67 ft

General:

$$k_E = \frac{307}{384}\, k_1 = \frac{307}{384}\,(980) = 781 \text{ kips/ft}$$

$$y_E = \frac{12.9}{781} = 0.0165 \text{ ft}$$

$$y_m = \alpha\beta\, y_E = 5(0.0165) = 0.0825 \text{ ft}$$

$$T_n = 2\pi\sqrt{\frac{K_{IM}m}{k_E}} = 6.28\sqrt{\frac{0.77(0.0109)}{781}}$$

$$= 0.0206 \text{ sec}$$

The basic equation for the numerical integration in table 7.3 is:

$$y_{n+1} = \ddot{y}_n(\Delta t)^2 + 2y_n - y_{n-1} \quad \text{(table 5.3)}$$

where

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(\Delta t)^2}{K_{IM}(m)} = \frac{(P_n - R_n)(0.00171)^2}{K_{IM}(m)}$$

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(0.00171)^2}{0.77(0.0109)} = 3.48(10^{-4})(P_n - R_n) \text{ ft, elastic range}$$

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(0.00171)^2}{0.78(0.0109)} = 3.439(10^{-4})(P_n - R_n) \text{ ft, elasto-plastic range)}$$

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(0.00171)^2}{0.66(0.0109)} = 4.064(10^{-4})(P_n - R_n) \text{ ft, plastic range}$$

Table 7.3. Determination of Maximum Deflection and Dynami
Reactions for Roof Slab

t (sec)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft
0	9.6	0	4.80	0.00167	0
0.00171	9.55	1.64	7.91	0.00275	0.001
0.00342	9.50	5.97	3.53	0.00123	0.006
0.00513	9.45	9.90	-0.45	-0.00015	0.011
0.00685	9.40	10.98	-1.58	-0.00054	0.017
0.00856	9.35	11.95	-2.60	-0.00089	0.022
0.01027	9.30	12.90	-3.60	-0.00146	0.026
0.01198	9.25	12.90	-3.65	-0.00148	0.028
0.01370	9.20	12.90	-3.70	-0.00150	0.030
0.01541	9.16	12.54	-3.38	-0.00118	0.029
0.01712	9.11	11.02	-1.91	-0.00066	0.028

* $(y_n)_{max}$ = 0.03 ft.

The time interval Δt = 0.00171 sec is approximately T_I
(par. 5-08). The value used is $t_o/40$ because the incident
data is presented in terms of t_o (EM 1110-345-413).

The dynamic reaction equations are listed in paragrap
values for the second column are obtained from figure 7.8,
144(6.67)/1000 = 0.96.

The maximum deflection $(y_n)_{max}$ computed in table 7.3
is less than the allowable y_m of 0.08 ft. This is satisfac
thin slabs are very sensitive. Note the variation in E a
two trials (pars. 7-22e and h).

$$\alpha\beta = \frac{(y_n)_{max}}{y_E} = \frac{0.03}{0.0165} = 1.8; \text{ OK}$$

k. <u>Shear Strength and Bond Stress.</u>

V_{max} = 6.02 kips (table 7.3)

For no shear reinforcement

Allowable $v_y = 0.04f'_c + 5000p$ (eq 4.24a)

$$v_y = 0.04(3000) + 5000(0.016) = 120 + 80 = 200 \text{ psi}$$

$$v = \frac{8V}{7bd} = \frac{8(6020)}{7(12)(2.75)} = 209 \text{ psi}$$

Slightly overstressed, OK to use

$$u = \frac{8V}{7\Sigma o d} = \frac{8(6020)}{7(5.7)2.75} = 440 \text{ psi}$$

Allowable $u = 0.15f'_c = 0.15(3000) = 450$ psi; OK

1. Summary.

3-3/4-in. slab, #3 at 2-1/2 in.

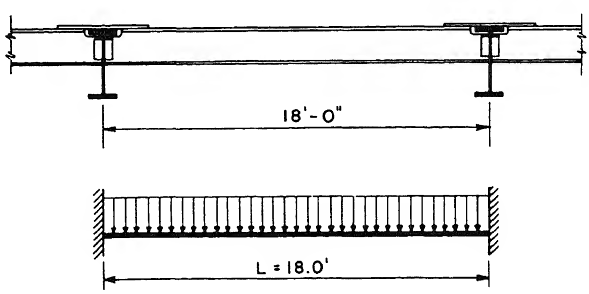
$p = 0.016$

No shear reinforcement

7-23 DESIGN OF ROOF PURLINS. The purlins are framed flush with the tops of the girders and are provided with moment-resisting connections. Connections attached to the top flanges of the purlins are embedded in the concrete slab to provide lateral support to the top compression flange and to prevent separation of slab from purlin in reversals.

Although composite behavior of the slab and purlin can be expected to develop to a limited extent, preliminary computations showed that design for independent behavior of slab and purlin is more desirable for this arrangement of members (pars. 4-12 and 6-23).

The purlins are designed for plastic behavior so that hinges are considered to develop at midspan and at the supports. In the design procedures of this manual only single-span elements can be handled. Therefore, the continuity of the purlins is accounted for approximately by designing interior purlins as fixed-end beams spanning 18 ft between girder centerlines.



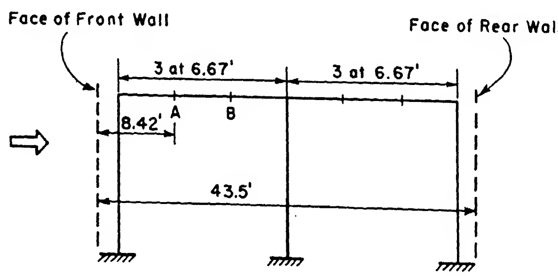
Depending on the exterior support condition the exterior purlins are designed as fixed-pinned beams or as fixed-fixed beams. In this example a typical interior purlin is designed.

a. Loading. To present a complete picture of the loads that need be considered acting on purlins, two directions for the blast wave are considered (par. 3-09).

For the blast wave moving normal to the long axis of the bu thus normal to the axis of the purlin, the loading may be conside uniformly distributed along the length of the purlin. For this c the pressure vs time variation at each point along the roof is a of its position (par. 3-09). In addition the load on a purlin is tion of the length of the slab spans because the load on the purl up to a maximum value in the time required for the blast wave to the two adjoining slabs. In the preliminary design of the purlin sign load is the simplest form of the roof load obtained from the overpressure vs time curve. The rise time, slab dynamic reaction local variation are all neglected in this preliminary step.

For the blast wave moving parallel to the long axis of the and thus parallel to the axis of the purlin, the load varies alon as a result of the time required for the blast wave to traverse t span. At any point along the purlin the time variation of the lo same and defined by the incident overpressure vs time curve.

In the calculations that follow the load vs time curves for lin are obtained first for the blast wave moving parallel to the of the building and then for the blast wave moving parallel to th axis of the building.



The purlin obtained preliminary design proced analyzed for both loads i tables 7.5, 7.6, and 7.7.

Blast wave moving perpend the long axis of the buil

From the procedure graph 3-09d the data for point (A) at purlin (A) are:

$$t_d = \frac{L'}{U_o} = \frac{8.42}{1403} = 0.006 \text{ sec}$$

$$v = \left[0.042 + (0.108)\,\frac{6.67}{43.5}\right] 1403 = 88.1 \text{ fps}$$

$$t_m = \frac{L'}{v} = \frac{8.42}{88.1} = 0.096 \text{ sec}$$

$$0.5(t_d + t_m) = 0.5(0.006 + 0.096) = 0.051 \text{ sec}$$

7-23a

$$t_m + 15\frac{h}{U_o} = 0.096 + 15\frac{16}{1403} = 0.266 \text{ sec}$$

$$4\left(\frac{P_{so}}{14.7}\right)\left(\frac{L'}{L} - 1\right) + 1 = 4\left(\frac{10}{14.7}\right)\left(\frac{8.42}{43.5} - 1\right) + 1 = -1.2$$

This value must be ≥ 0; it is therefore taken as zero.

The resulting variations in the ratio of local roof overpressure to incident overpressure for both points (A) and (B) are plotted in figure 7.15. The calculations for point (B) are not shown but are similar to those for point (A).

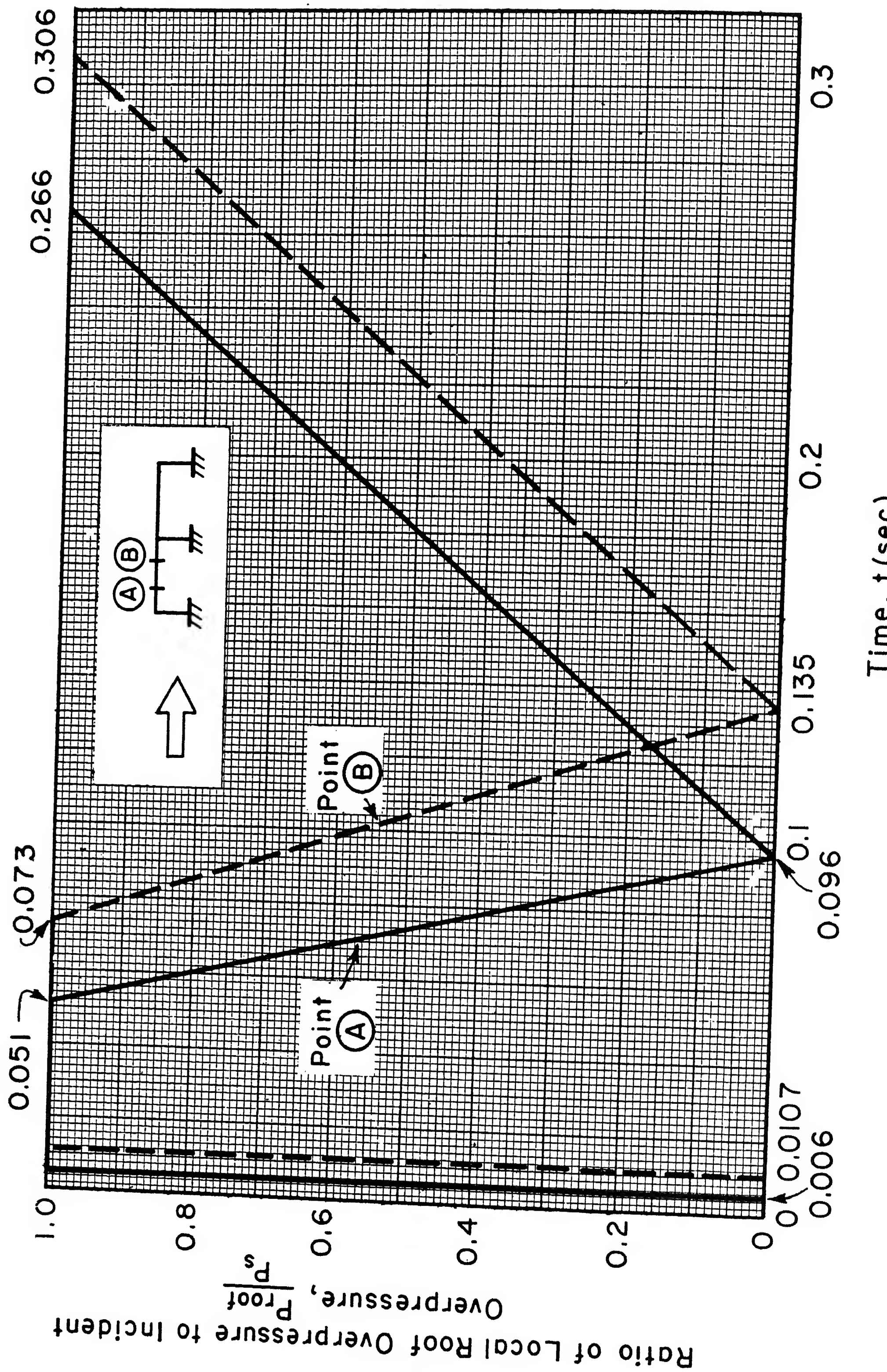
By combining figure 7.15 and the incident overpressure curve (fig. 7.8) the variation of local roof overpressure with time is determined (fig. 7.16) for points (A) and (B). The calculations of local roof overpressure for point (A) are contained in table 7.4. The calculations of data for point (B) are not shown.

Table 7.4. Computation of Local Roof Overpressure at Point (A)

t (sec)	$t - t_d$ (sec)	$\frac{t - t_d}{t_o}$	$\frac{P_s}{P_o}$ (fig. 7.15)	P_s (fig. 7.8)	$\frac{P_{roof}}{P_s}$	P_{roof} (psi)
0.00475	0	0	1.0	10.0	1.0	10.0
0.0429	0.03815	0.0558	0.893	8.93	1.0	8.93
0.07325	0.0685	0.1	0.814	8.14	0.205	1.67
0.0810	0.07625	0.1115	0.795	7.95	0.0	0.0
0.14175	0.1370	0.20	0.655	6.55	0.380	2.49
0.21025	0.2055	0.30	0.519	5.19	0.810	4.20
0.24100	0.23625	0.346	0.463	4.63	1.0	4.63
0.27875	0.274	0.4	0.402	4.02	1.0	4.02
0.34725	0.3425	0.5	0.303	3.03	1.0	3.03
0.41575	0.411	0.6	0.220	2.20	1.0	2.20
0.48375	0.479	0.7	0.149	1.49	1.0	1.49
0.55275	0.548	0.8	0.090	0.90	1.0	0.90
0.62075	0.616	0.9	0.041	0.41	1.0	0.41
0.68975	0.685	1.0	0.0	0.0	1.0	0.0

The data in the last column are plotted in figure 7.16.

The roof slab is analyzed in table 7.5 for both local overpressure vs time curves presented in figure 7.16. The analysis in table 7.5 is based


Ratio of Local Roof Overpressure to Incident Overpressure, $\frac{P_{roof}}{P_s}$
Time, t (sec)
Point A
Point B
A
B
0
0.2
0.4
0.6
0.8
1.0
0
0.006
0.0107
0.051
0.073
0.096
0.1
0.135
0.2
0.266
0.3
0.306

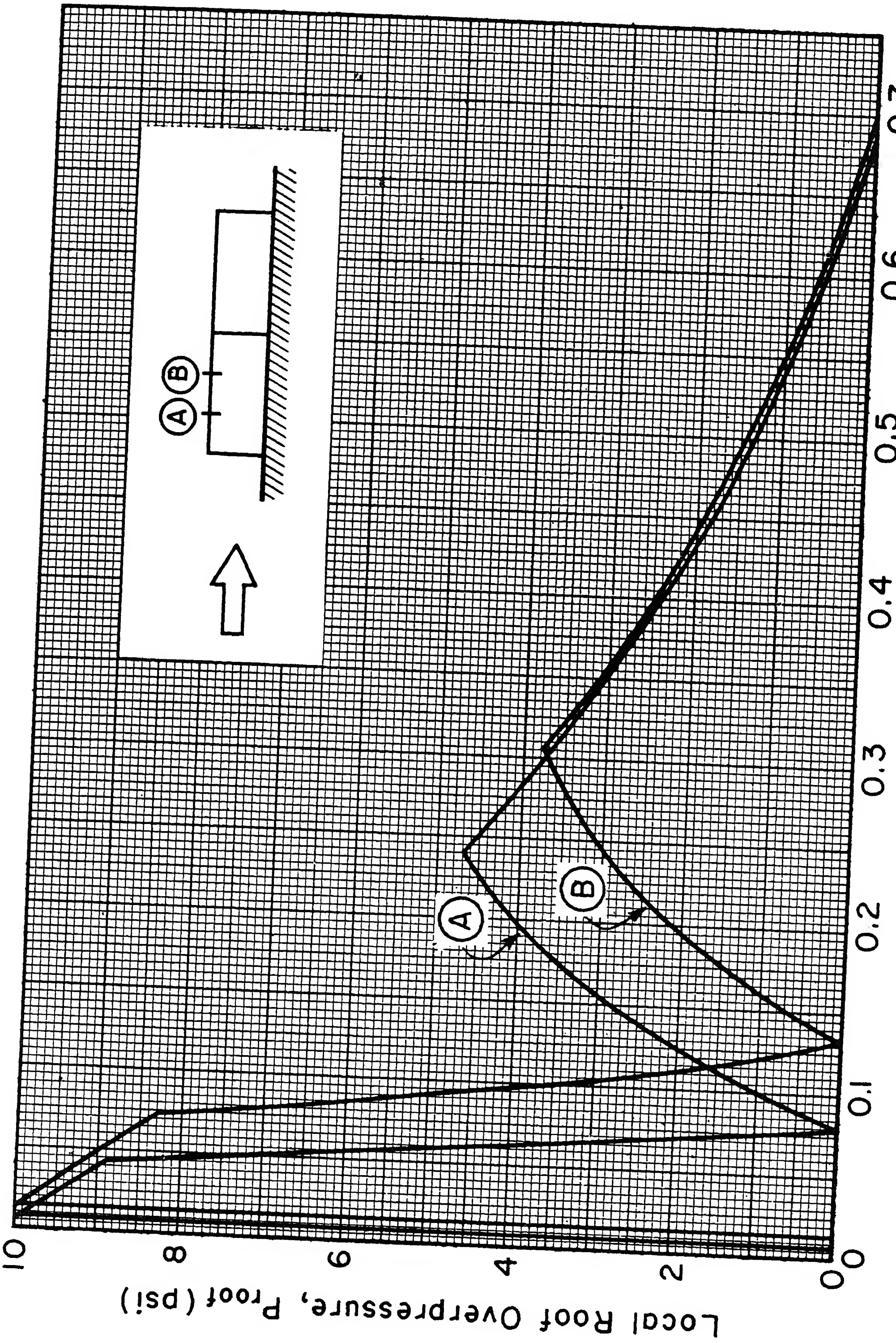


Figure 7.16. Local roof overpressure vs time for points (A) and (B)

Table 7.5. Determination of Dynamic Reactions for Roof Slab, Local Roof Overpressure at Purlins (A) and (B)

	Purlin (A)						Purlin (B)	
t (sec)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)	V_n* (kips)	P_n (kips)	V (ki
0	0	0	0.62	+0.00030	0	0	0	0
0.002	3.97	0.29	3.68	0.00175	0.00030	0.66	3.97	0.
0.004	7.94	2.30	5.64	0.00268	0.00235	1.94	7.94	1
0.006	9.47	6.94	2.53	0.00120	0.00708	3.83	9.47	3
0.008	9.42	10.15	-0.73	-0.00034	0.01301	5.00	9.42	5
0.010	9.36	11.24	-1.88	-0.00088	0.01860	5.41	9.36	5
0.012	9.30	12.17	-2.87	-0.00134	0.02331	5.77	9.30	5
0.014	9.25	12.83	-3.58	-0.00168	0.02668	6.02	9.25	6
0.016	9.20	12.90	-3.70	-0.00206	0.02837	6.00	9.20	6
0.018	9.14	12.54	-3.40	-0.00162	0.02800	5.79	9.14	5
0.020	9.09	10.59	-1.50	-0.00071	0.02601	5.08	9.09	5
0.022	9.03	7.94			0.02331	4.51	9.03	4
0.024	8.98					4.49	8.98	4
0.026	8.93					4.46	8.93	4
0.028	8.88					4.44	8.88	4
0.030	8.82					4.41	8.82	4
0.032	8.76					4.38	8.76	4
0.034	8.70					4.35	8.70	4
0.036	8.64					4.32	8.64	4
0.038	8.60					4.30	8.60	4
0.040	8.56					4.28	8.54	4
0.042	8.10					4.05	8.50	4
0.044	7.64					3.82	8.44	4
0.046	7.18					3.59	8.38	4
0.048	6.70					3.35	8.34	4
0.050	6.24					3.12	8.28	4
0.052	5.78					2.89	8.22	4
0.054	5.32					2.66	8.18	4
0.056	4.86					2.43	8.12	4
0.058	4.38					2.19	8.08	4
0.060	3.88					1.94	8.02	4
0.062	3.46					1.73	7.96	3
0.064	3.00					1.50		
0.066	2.54					1.27		
0.068	2.06					1.03		

* From t = 0.020 the values of V_n are equal to $0.50P_n$.

in the following data developed in paragraph 7-22j and the resistance diagram for the roof slab (fig. 7.14):

Elastic range: $\ddot{y}_n(\Delta t)^2 = 3.48(10^{-4})(P_n - R_n)$ ft

Elasto-plastic range: $\ddot{y}_n(\Delta t)^2 = 3.439(10^{-4})(P_n - R_n)$ ft

Plastic range: $\ddot{y}_n(\Delta t)^2 = 4.064(10^{-4})(P_n - R_n)$ ft

The object of this computation is to determine the slab dynamic reactions on the purlins.

The maximum response of the slab to these loads occurs before there is any difference between the loads at (A) and (B), thus the dynamic reactions of the slabs at purlins (A) and (B) are the same until the dynamic reactions are based on the applied load P above. The last two columns of table 7.5 show the applied load and the dynamic reactions at purlin (B).

To obtain the design load for purlins (A) and (B) the dynamic reaction data from table 7.5 are plotted in figure 7.17. The total purlin load is equal to the sum of the reactions of the slabs forward and aft of the purlin. In figure 7.17, it may be seen that the same dynamic reactions are plotted with a time lag

$$t_{lag} = \frac{6.67}{1403} = 0.0048 \text{ sec}$$

The loads from figure 7.17 are used in tables 7.6 and 7.7 (par. 7-23j) to check the preliminary purlin design.

Blast wave moving parallel to the long axis of the building:

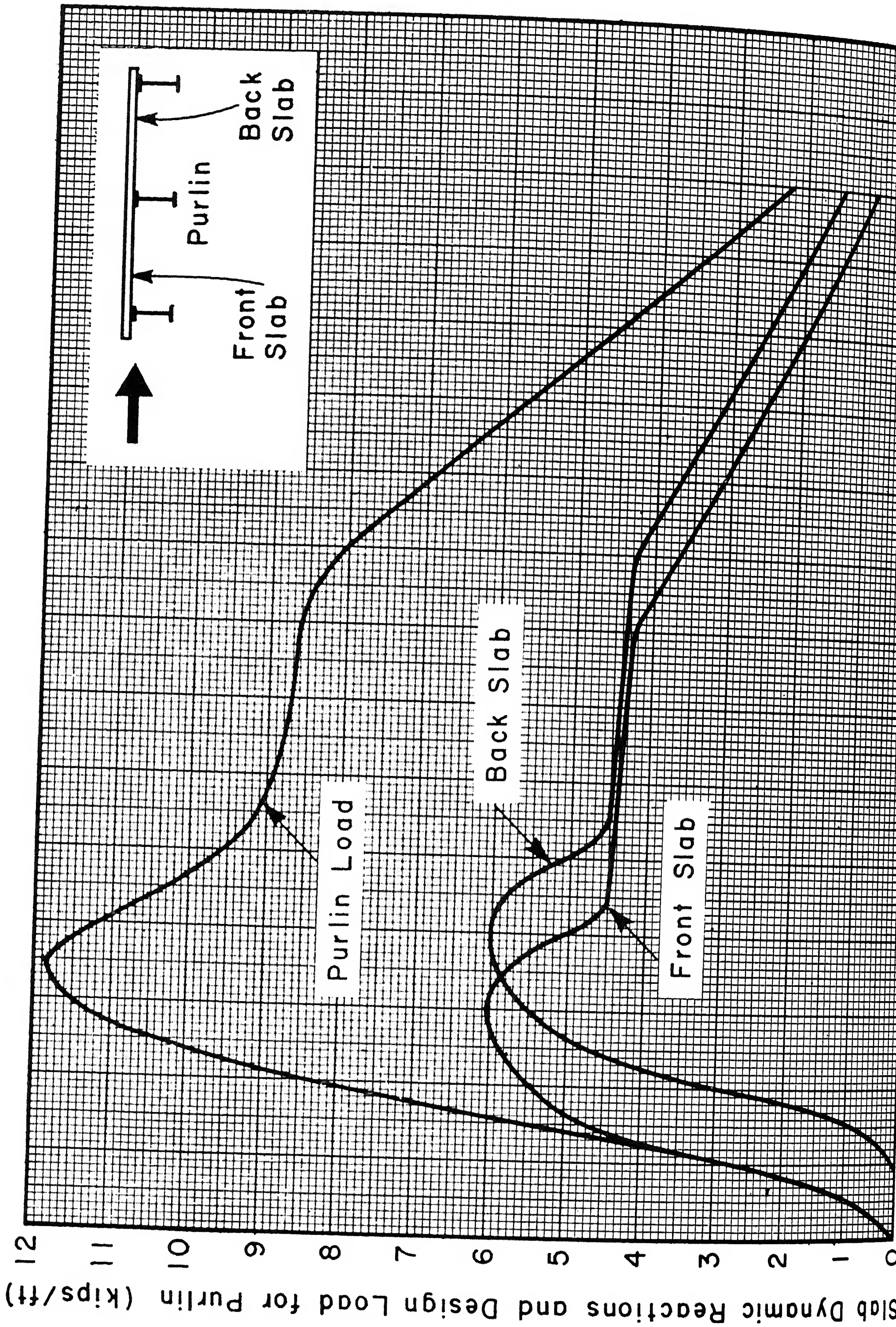
The design load on the purlin is determined in figure 7.17a using the slab dynamic reactions obtained in table 7.3 for incident overpressure. The variation of the average load on the purlin with time is found by plotting the same dynamic reaction curve with a time lag

$$t_d = \frac{18}{1403} = 0.0128 \text{ sec}$$

The variation in slab dynamic reaction with time is the same at each point along the purlin. The load curve from figure 7.17a is used in table 7.8 to check the preliminary purlin design.

Preliminary design:

For preliminary design it is desirable to use a simple load-time


Back Slab
Purlin
Front Slab
Purlin Load
Back Slab
Front Slab
0 1 2 3 4 5 6 7 8 9 10 11 12
Slab Dynamic Reactions and Design Load for Purlin (kips/ft)

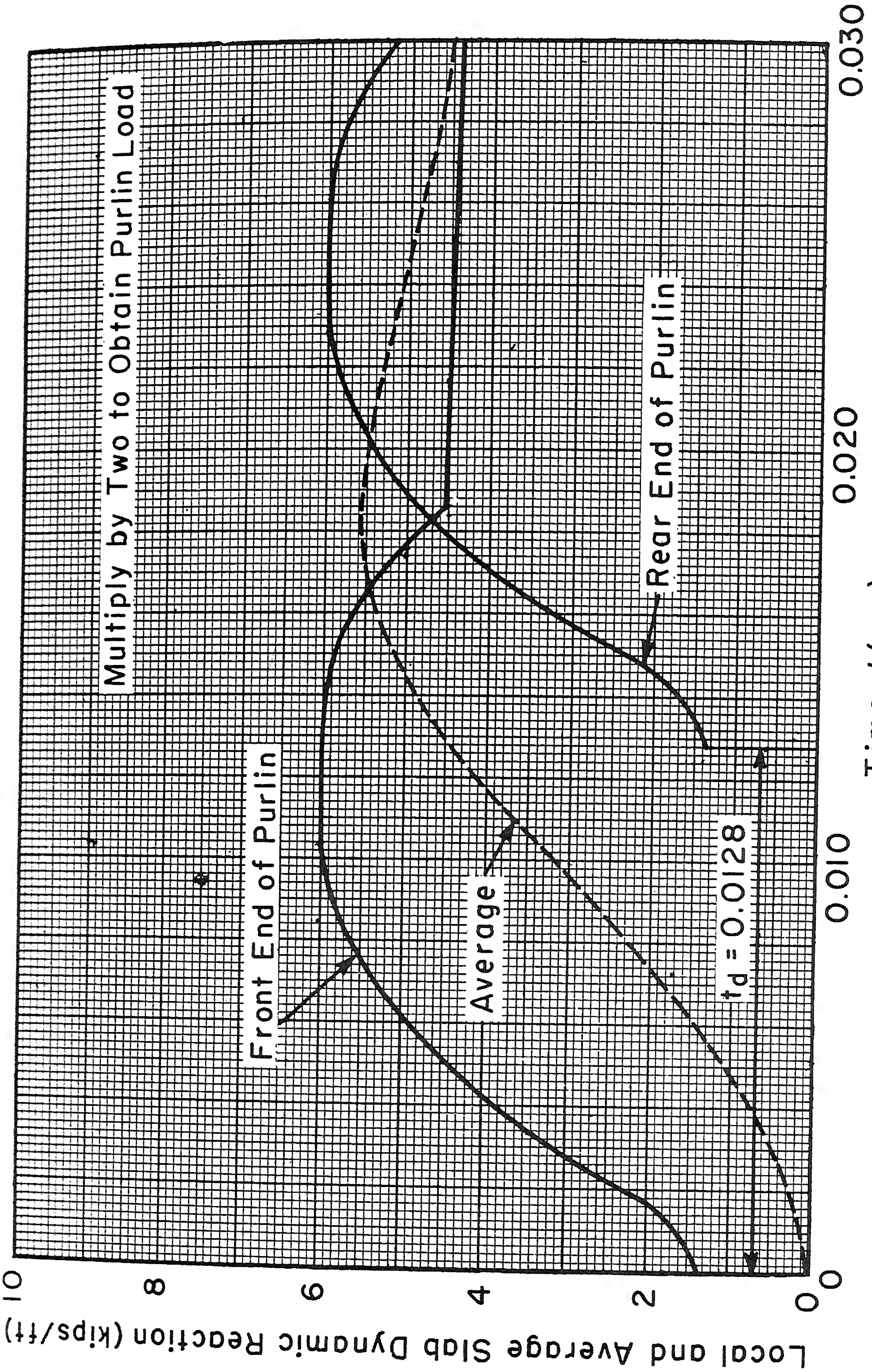
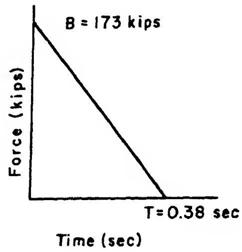


Figure 7.17a. Design loads for any purlin for incident overpressure

curve. The design load as idealized from the computed loading sh
figure 7.8 is defined by



$$B = 10 \text{ psi} = \frac{10(144)6.67(18)}{1000} = 173 \text{ kips}$$

$$T = 0.38 \text{ sec}$$

$$H = \frac{BT}{2} = \frac{(173)0.38}{2} = 32.9 \text{ kip-sec (par. 6-11)}$$

b. Dynamic Design Factors. (Refer to table 6.1.)

Elastic range:

$$K_L = 0.53, \quad K_M = 0.41, \quad K_{LM} = 0.77$$

$$R_{1m} = \frac{12M_{Ps}}{L}$$

$$V_1 = 0.36R + 0.14P, \quad k_1 = \frac{384EI}{L^3}$$

Elasto-plastic range:

$$K_L = 0.64, \quad K_M = 0.50, \quad K_{LM} = 0.78$$

$$R_m = \frac{8}{L}(M_{Ps} + M_{Pm}), \quad k_{ep} = \frac{384EI}{5L^3}$$

$$V = 0.39R + 0.11P$$

Plastic range:

$$K_L = 0.50, \quad K_M = 0.33, \quad K_{LM} = 0.66$$

$$R_m = \frac{8}{L}(M_{Ps} + M_{Pm})$$

$$V = 0.38R_m + 0.12P$$

Average values:

$$K_L = 0.5(0.64 + 0.50) = 0.57$$

$$K_M = 0.5(0.50 + 0.33) = 0.42$$

$$K_{LM} = 0.5(0.78 + 0.77) = 0.77$$

$$R_m = \frac{8}{L}(M_{Ps} + M_{Pm})$$

$$k_E = \frac{307EI}{L^3}$$

c. First Trial - Actual Properties.

$$M_{Ps} \equiv M_{Pm} = M_P$$

Let $\alpha\beta = 6$ (par. 6-26)

Assume $C_R = 1.0$ (experience)

$R_m = C_R B = 1.0(173) = 173$ kips

$M_p = 1.05 S f_{dy} = \dfrac{1.05(41.6)S}{12} = 3.64S$ kip-ft (S in in.^{3}) (eq 4.2a)

$R_m = \dfrac{16M_p}{L} = \dfrac{(16)3.64S}{18} = 173, \therefore S = 53.5$ in.^{3}

Try 16 WF 36,

$S = 56.3$ in.^{3}, $Z = 64$ in.^{3}, $0.5(S + Z) = 60.1$ in.^{3}, $I = 446$ in.^{4}

$M_p = f_{dy}\left(\dfrac{S + Z}{2}\right) = \dfrac{41.6(60.1)}{12} = 208$ kip-ft (eq 4.2)

$R_m = \dfrac{16M_p}{L} = \dfrac{16(208)}{18} = 185$ kips

$k_E = \dfrac{307EI}{L^3} = \dfrac{(307)30(10)^3 446}{18^3(144)} = 4900$ kips/ft

$y_E = \dfrac{R_m}{k_E} = \dfrac{185}{4900} = 0.0378$ ft

$y_m = \alpha\beta\, y_E = 6(0.0378) = 0.2268$ ft (par. 6-26)

$\text{Weight} = \left[\dfrac{3.75(150)}{(12)} + 6.0\right]\dfrac{6.67(18)}{1000} + \dfrac{36(18)}{1000} = 6.35 + 0.65 = 7.0$ kips

Mass $m = \dfrac{7.0}{32.2} = 0.217$ kip-sec^2/ft

d. First Trial - Equivalent System Properties.

$R_{me} = K_L R_m = 0.57(185) = 105.5$ kips (eq 6.12)

$H_e = K_L H = 0.57(32.9) = 18.75$ kip-sec (eq 6.2)

$m_e = K_M m = 0.42(0.217) = 0.091$ kip-sec^2/ft (eq 6.18)

$W_p = \dfrac{(H_e)^2}{2m_e} = \dfrac{(18.75)^2}{2(0.091)} = 1930$ ft-kips

$T_n = 2\pi\sqrt{K_{LM} m/k_E} = 6.28\sqrt{0.77(0.217)/4900} = 0.0368$ sec

e. Work Done vs Energy Absorption Capacity.

$C_T = T/T_n = 0.38/0.0368 = 10.3$

$C_R = R_m/B = 185/173 = 1.07$ (eqs 6.15, 6.16)

$t_m/T = 0.12$ (fig. 5.29)

$t_m = (0.12)0.38 = 0.0455$ sec

Idealized load-time curve is satisfactory at $t = 0.045$ sec

$C_W = 0.008$ (fig. 5.27)

$W_m = C_W W_P = 0.008(1930) = 15.4$ ft-kips (eq 6.17)

$E = R_{me}(y_m - 0.5y_E) = 105.5\ [0.2268 - 0.5(0.0378)]$

$= 22.0$ ft-kips (eq 6.18)

$E > W$, therefore the selected proportions are satisfactory
liminary design.

Try to bring E closer to W by another trial that follow

f. <u>Second Trial - Actual Properties.</u>

$$R_m = \frac{0.25W_m + 0.75E}{K_L(y_m - 0.5y_E)} = \frac{0.25(15.4) + 0.75(22.0)}{(0.57)[0.2268 - 0.5(0.0378)]} = 172 \text{ ki}$$

Since $R_m = 172 \approx 173$ from first trial, try

$$R_m = \frac{0.5W_m + 0.5E}{K_L(y_m - 0.5y_E)} = \frac{0.5(15.4 + 22.0)}{0.57(0.208)} = 158 \text{ kips}$$

$$R_m = \frac{16M_P}{L} = \frac{16(3.64S)}{18} = 158, \therefore S = 48.5$$

Try 14 WF 34,

$S = 48.5$ in.^{3}, $Z = 54.5$ in.^{3}, $0.5(S + Z) = 51.5$ in.^{3}, I =

$$M_P = f_{dy}\frac{(S + Z)}{2} = \frac{41.6(51.5)}{12} = 178 \text{ kip-ft (eq 4.2)}$$

$$R_m = \frac{16M_P}{L} = \frac{16(178)}{18} = 159 \text{ kips}$$

$$k_E = \frac{307EI}{L^3} = \frac{307(30)10^3(339.2)}{18^3(144)} = 3720 \text{ kips/ft}$$

$$y_E = \frac{R_m}{k_E} = \frac{159}{3720} = 0.0427 \text{ ft}$$

$y_m = \alpha\beta\ y_E = 6(0.0427) = 0.2562$ (par. 6-26)

g. <u>Second Trial - Equivalent System Properties.</u>

$R_{me} = K_L R_m = 0.57(159) = 90.5$ kips (eq 6.12)

$k_e = K_L k_E = 0.57(3720) = 2120$ kips/ft (eq 6.6)

$H_e = K_L H = 0.57(32.9) = 18.75$ kip-sec (eq 6.2)

$m_e = K_M m = 0.42(0.217) = 0.091$ kip-sec^2/ft (eq 6.8)

$$W_p = \frac{H_e^2}{2m_e} = \frac{(18.75)^2}{2(0.091)} = 1930 \text{ ft-kips}$$

$$T_n = 2\pi\sqrt{K_{LM}m/k_E} = 6.28\sqrt{0.77(0.217)/3720} = 0.042 \text{ sec}$$

h. Work Done vs Energy Absorption Capacity.

$$C_T = T/T_n = 0.38/0.042 = 9.05$$

$$C_R = R_m/B = 159/173 = 0.92$$

$$C_W = 0.022$$

$W_m = 0.022(1930) = 42.5$; too large, use initial trial 16 W 36

i. Preliminary Design for Shear Stress.

Estimated $V_{max} = 0.5R_m = 0.5(185) = 92.5$ kips

$$v = \frac{V}{t_w d} = \frac{92{,}500}{0.299(15.85)} = 19{,}500 \text{ psi}$$

Allowable v = 21,000 psi; OK (par. 4-05c)

j. Determination of Maximum Deflection and Dynamic Reactions by Numerical Integration.

16 W 36,

$S = 56.3 \text{ in.}^3$, $Z = 64 \text{ in.}^3$, $0.5(S + Z) = 60.1 \text{ in.}^3$, $I = 446 \text{ in.}^4$

$M_p = 208$ kip-ft

Weight = 7.0 kips

Mass m = 0.217 kip-sec^2/ft

Elastic range:

$$R_{1m} = \frac{12M_p}{L} - \text{weight} = \frac{12(208)}{18} - 7.0 = 132.0 \text{ kips}$$

$$k_1 = \frac{384EI}{L^3} = \frac{(384)30(10)^3 446}{18^3(144)} = 6130 \text{ kips/ft}$$

$$y_e = \frac{R_{1m}}{k_1} = \frac{132.0}{6130} = 0.0215 \text{ ft}$$

Elasto-plastic range:

$$R_m = \frac{16M_p}{L} - \text{weight} = \frac{16(208)}{18} - 7.0 = 178.0 \text{ kips}$$

$$k_{ep} = \frac{384EI}{5L^3} = \frac{1}{5}k_1 = 1226 \text{ kips/ft}$$

$$y_{ep} = y_e + \frac{R_m - R_{1m}}{k_{ep}} = 0.0215 + \frac{178 - 132}{1226} = 0.0215 + 0.037$$

Plastic range:

$$R_m = \frac{16M_P}{L} - \text{weight} = 178.0 \text{ kips}$$

$$k_E = \frac{307EI}{L^3} = 4900 \text{ kips/ft}$$

$$y_E = \frac{R_m}{k_E} = \frac{178.0}{4900} = 0.0363 \text{ f}$$

$$y_m = \alpha\beta\, y_E = 6(0.0363) = 0$$

The basic equation f
numerical integration in t
7.7, and 7.8 is $y_{n+1} = \ddot{y}$
$2y_n - y_{n-1}$ (table 5.3)

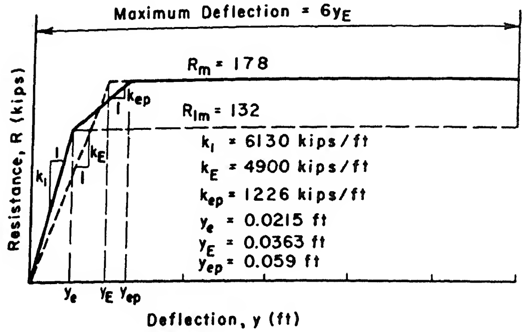


Figure 7.18. Resistance function for 16 WF 36 purlin spanning 18 ft

Table 7.6. Determination of Maximum Deflection and Dynamic Reactions for Purl (Zone 3 Local Roof Overpressure)

t (sec)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)
0	0	0	3.3	0.00018	
0.003	20.7	1.1	19.6	0.00106	0.00018
0.006	73.8	9.0	64.8	0.00349	0.00142
0.009	129.6	38.8	90.8	0.00489	0.00615
0.012	180.0	99.5	80.5	0.00434	0.01577
0.015	207.0	142.1	64.9	0.00345	0.02973
0.018	212.4	163.4	49.0	0.00260	0.04714
0.021	192.6	178.0	14.6	0.00092	0.06715
0.024	176.4	178.0	-1.6	-0.00010	0.08808
0.027	163.8	178.0	-14.2	-0.00089	0.10891
0.030	160.2	178.0	-17.8	-0.00112	0.12885
0.033	158.4	178.0	-19.6	-0.00123	0.14767
0.036	156.6	178.0	-21.4	-0.00134	0.16526
0.039	154.8	178.0	-23.2	-0.00146	0.18151
0.042	151.2	178.0	-26.8	-0.00168	0.19630
0.045	145.8	178.0	-32.2	-0.00202	0.20941
0.048	131.4	178.0	-46.6	-0.00293	0.22050
0.051	118.8	178.0	-59.2	-0.00372	0.22866
0.054	104.4	178.0	-73.6	-0.00463	0.23310
0.057	93.6	176.8	-83.2	-0.00448	0.23291
0.060	81.0	147.3	-66.3	-0.00357	0.22824
0.063	68.4	95.3	-26.9	-0.00145	0.22000
0.066	55.8	34.2			0.21031
0.069	45.0				

* $(V_n)_{max}$ = 90.8 kips.

Table 7.7. Determination of Maximum Deflection and Dynamic Reactions for Purlin (B)
(Zone 3 Local Roof Overpressure)

t (sec)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)	V_n (kips)
0	0	0	3.3	0.00018	0	0
0.003	20.7	1.1	19.6	0.00106	0.00018	3.3
0.006	73.8	9.0	64.8	0.00349	0.00142	13.6
0.009	129.6	38.8	90.8	0.00489	0.00615	32.1
0.012	180.0	99.5	80.5	0.00434	0.01577	61.0
0.015	207.0	142.1	64.9	0.00345	0.02973	78.1
0.018	212.4	163.4	49.0	0.00260	0.04714	87.1
0.021	192.6	178.0	14.6	0.00092	0.06715	90.8*
0.024	176.4	178.0	-1.6	-0.00010	0.08808	88.8
0.027	163.8	178.0	-14.2	-0.00089	0.10891	87.3
0.030	160.2	178.0	-17.8	-0.00112	0.12885	86.8
0.033	158.4	178.0	-19.6	-0.00123	0.14767	86.6
0.036	156.6	178.0	-21.4	-0.00134	0.16526	86.4
0.039	154.8	178.0	-23.2	-0.00146	0.18151	86.2
0.042	153.5	178.0	-24.5	-0.00154	0.19630	86.1
0.045	152.1	178.0	-25.9	-0.00163	0.20955	85.9
0.048	150.7	178.0	-27.3	-0.00172	0.22117	85.7
0.051	149.3	178.0	-28.7	-0.00180	0.23107	85.6
0.054	147.9	178.0	-30.1	-0.00189	0.23917	85.4
0.057	146.5	178.0	-31.5	-0.00198	0.24538	85.2
0.060	145.1	178.0	-32.9	-0.00207	0.24961	85.1
0.063	143.7	178.0	-34.3	-0.00216	0.25177	84.9
0.066	142.3	178.0	-35.7	-0.00224	0.25177	84.7
0.069	140.9	163.9	-23.0	-0.00124	0.24953	78.7
0.072	139.5	141.9			0.24605	70.6

* $(V_n)_{max}$ = 90.8 kips.

Table 7.8. Determination of Maximum Deflection and Dynamic Reactions for any Purlin
(Incident Overpressure)

t (sec)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)	V_n (kips)
0	0	0	2.8	0.00015	0	
0.003	18.0	0.9	17.1	0.00092	0.00015	
0.006	50.4	7.5	42.9	0.00231	0.00122	
0.009	97.2	28.2	69.0	0.00372	0.00460	
0.012	147.6	71.7	75.9	0.00409	0.01170	
0.015	183.6	133.7	49.9	0.00265	0.02289	
0.018	201.6	150.7	50.9	0.00271	0.03673	
0.021	194.4	171.0	23.4	0.00124	0.05328	
0.024	183.6	178.0	5.6	0.00035	0.07107	89.7
0.027	172.8	178.0	-5.2	-0.00033	0.08921	
0.030	162.0	178.0	-16.0	-0.00101	0.10702	
0.033	160.6	178.0	-17.4	-0.00109	0.12382	
0.036	159.2	178.0	-18.8	-0.00118	0.13953	
0.039	157.8	178.0	-20.2	-0.00127	0.15406	
0.042	156.5	178.0	-21.5	-0.00135	0.16732	
0.045	155.1	178.0	-22.9	-0.00144	0.17923	
0.048	153.7	178.0	-24.3	-0.00153	0.18970	
0.051	152.3	178.0	-25.7	-0.00161	0.19864	
0.054	150.9	178.0	-27.1	-0.00170	0.20597	
0.057	149.5	178.0	-28.5	-0.00179	0.21160	
0.060	148.2	178.0	-29.8	-0.00187	0.21544	
0.063	146.8	178.0	-31.2	-0.00196	0.21741	
0.066	145.4	178.0	-32.6	-0.00205	0.21742	
0.069					0.21538	

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(\Delta t)^2}{K_{LM}(m)}$$

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)9(10^{-6})}{0.77(0.217)} = 5.386(10^{-5})(P_n - R_n) \text{ ft, elastic}$$

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)9(10^{-6})}{0.78(0.217)} = 5.317(10^{-5})(P_n - R_n) \text{ ft, elasto-p range}$$

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)9(10^{-6})}{0.66(0.217)} = 6.284(10^{-5})(P_n - R_n) \text{ ft, plastic}$$

The time interval $\Delta t = 0.003$ sec is approximately $T_n/10 = 0.00$ (par. 5-08).

The dynamic reaction equations are listed in paragraph 7-23b. values for tables 7.6 and 7.7, second column, are obtained from figu multiplying by 18 to account for the length of the purlin. For tabl the P_n values in the second column are obtained from figure 7.17a mu ing by 2 × 18 = 36 to account for the two slabs loading the purlin.

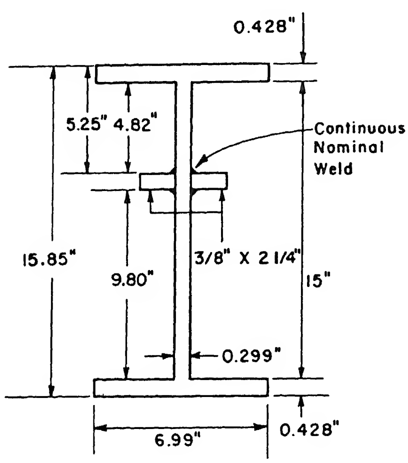
The maximum deflection $(y_n)_{max}$ computed in table 7.7 is 0.252 Thus $\alpha\beta = \frac{0.252}{0.0363} = 6.9 > 6$. This is satisfactory for this purpose. other purlins are less critical.

k. Shear Stress Check.

$V_{max} = 90.8$ kips (tables 7.6 and 7.7)

$$v = \frac{V}{dt_w} = \frac{90,800}{15.85(0.299)} = 19,200 \text{ psi}$$

Allowable v = 21,000 psi; OK (par. 4-05c)



l. Check Proportions for Loca Buckling. (par. 4-06d)

16 W= 36, $b = 6.992$, $t_f = 0.428$,

$a = 15.0$, $t_w = 0.299$

$b/t_f = 6.99/0.428 = 16.4 > 14.0$, OK; slab provides support

$a/t_w = 15.0/0.299 = 50.2 > 30$, NG; tudinal stiffeners required

$t_s = 3/8 > 0.299$, OK

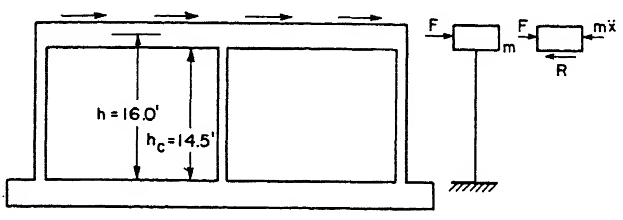
$b_s/t_s = 6$; $b_s = 6t_s = 6(3/8) = 2.2$

Use two plates 3/8 in. by 2-1/4 in. full length.

$c/t_w = (5.25 - 0.43)/0.299 = 16 < 22$; OK

$e/t_w = (10.25 - 0.43)/0.299 = 32.8 < 40$; OK

7-24 PRELIMINARY DESIGN OF COLUMNS. A single-story frame subject to lateral load behaves essentially as a single-degree-of-freedom system with the columns displaying the spring properties. It is therefore unnecessary to substitute an equivalent system for the original structure and the mass and load factors which are necessary in the design of beams and slabs are not used in the design of single-story frames.



The preliminary plastic design procedure of paragraph 6-11 is the basis for determining the preliminary column size. The equations of paragraph 7-06 are incorporated into the procedure of paragraph 6-11 replacing some of the factors that are used to determine equivalent systems.

For purposes of preliminary design the frame girders are assumed to be infinitely rigid thus simplifying the determination of the column spring constant. For spring constant computation the effective column height h is 16 ft based on an assumed girder depth of 3 ft and a clear height h_c of 14.5 ft. The clear height is used in determining the resistance of the columns (par. 7-06).

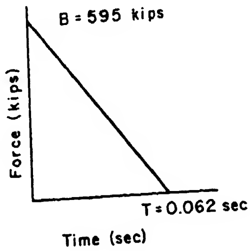
In the preliminary design of steel columns it is desirable that the energy absorption capacity be greater than the work done on the frame as an allowance for the factors which are neglected. These factors are: (1) the effect of direct stress on the plastic hinge moment, (2) the effect of lateral deflection of the column on its resistance, and (3) the effect of girder flexibility.

a. Design Loading. The design lateral load on the frame is obtained from the dynamic reactions at the top of the front wall slab. However, for preliminary design computations, it is satisfactory to use the net lateral overpressure curve (fig. 7.11).

The net lateral load is assumed to be reacted equally by the frame

and the foundation. This results in a conservative load for the f
since the dynamic reaction equations for the wall slab (par. 7-21b
in footing reactions that are larger than the roof reactions.

The design load as idealized from the co
loading as shown by figure 7.11 is defined by



$$B = 25.3 \text{ psi} = \frac{25.3(144)18\left(\frac{17.5}{2} + 0.31\right.}{1000}$$

$$T = 0.062 \text{ sec}$$

$$H = \frac{BT}{2} = \frac{595(0.062)}{2} = 18.45 \text{ kip-sec}$$

b. Mass Computation.

Walls $\frac{11(150)18(2)17.5}{12(1000)} = 86.5$ kips

Slab and roofing $\left(\frac{3.75(150)}{12} + 6.0\right) \frac{18(43.5)}{1000} = 41.4$ kips

Purlins $\frac{42.4(18)7}{1000} = 5.3$ kips

Girder (estimate) $\frac{160(41)}{1000} = 6.5$ kips

Connections (allow 10%) $= 0.1(5.3 + 6.5) = 1.2$ kips

Columns (estimate) $\frac{140(3)14.5}{1000} = 6.1$ kips

Mass of single-degree-of-freedom system = total roof + 1/
and walls)

$$m = \frac{41.4 + 5.3 + 6.5 + 1.2 + 0.33(86.5 + 6.1)}{32.2} = \frac{85.2}{32.2} = 2.64$$

c. First Trial - Actual Properties.

Let $\alpha\beta = 12$ (par. 6-26)

Assume $C_R = 0.5$ (experience)

$R_m = C_R B = 0.5(595) = 297$ kips

$M_p = 1.05 S f_{dy} = \frac{1.05(41.6)}{12} S = 3.64S$ kip-ft (S in in.^{3})(

$R_m = (2nM_p)/h_c = [2(3)3.64S]/14.5 = 1.51S = 297, \therefore S =$

Smallest column that satisfies buckling criteria is 14 W
(par. 4-06d)

$S = 216.0$ in.^{3}, $Z = 242.7$ in.^{3}, $I = 1593$ in.^{4}, $0.5(S +$

$$a/t_w = 12.63/0.66 = 19.2 < 22; \text{ OK}$$

$$b/t_f = 14.74/1.063 = 13.9 < 14.0; \text{ OK (par. 4-06d)}$$

$$M_P = 0.5(S + Z)f_{dy} = \frac{229.3(41.6)}{12} = 795 \text{ kip-ft}$$

$$R_m = \frac{2nM_P}{h_c} = \frac{2(3)795}{14.5} = 328 \text{ kips}$$

$$k = \frac{12EIn}{h^3} = \frac{12(30)10^3(1593)3}{16^3(144)} = 2910 \text{ kips/ft}$$

$$x_e = \frac{R_m}{k} = \frac{328}{2910} = 0.113 \text{ ft}$$

$$x_m = \alpha\beta\, x_e = 12(0.113) = 1.35 \text{ ft}$$

$$T_n = 2\pi\sqrt{m/k} = 6.28\sqrt{2.64/2910} = 0.189 \text{ sec}$$

d. First Trial - Work Done vs Energy Absorption Capacity.

$$T/T_n = 0.062/0.189 = 0.328$$

$$C_R = R_m/B = 328/595 = 0.55$$

$$t_m/T = 1.3, \; t_m = 1.3(0.062) = 0.0805 \text{ (fig. 5.29)}$$

The original load-time curve should be revised to obtain a closer approximation to the total impulse up to time t_m (par. 5-13). The impulse up to t = 0.09 sec in figure 7.11 is

$$H = 1.00 \text{ psi-sec (obtained by graphical integration)}$$

$$T = 2H/B = 2(1.0)/25.3 = 0.079 \text{ sec}$$

$$T/T_n = 0.079/0.188 = 0.42$$

$$t_m/T = 1.2, \; t_m = 1.2(0.079) = 0.095 \text{ (fig. 5.29)}$$

Try again for impulse up to t = 0.10 sec, H = 1.026

$$T = 2(1.026)/25.3 = 0.081 \text{ sec}$$

$$T/T_n = 0.081/0.189 = 0.43$$

$$t_m/T = 1.2, \; t_m = 1.2(0.081) = 0.097 \approx t = 0.10; \text{ OK (fig. 5.29)}$$

$$C_W = 0.71 \text{ (fig. 5.27)}$$

$$W_P = \frac{H^2}{2m} = \left(\frac{BT}{2}\right)^2/2m = \frac{(595)^2(0.081)^2}{8(2.64)} = 111 \text{ ft-kips}$$

$$W_m = C_W W_P = 0.71(111) = 78.7 \text{ ft-kips}$$

$$E = R_m\left(x_m - \frac{x_e}{2}\right) = 328\,[1.35 - 0.5(0.113)] = 426 \text{ ft-kips}$$

This column section (14 WF 136) is more than ample. Try to use a smaller column size.

e. Second Trial - Actual Properties.

$$R_m = \frac{0.5(W_m + E)}{y_m} = \frac{0.5(78.7 + 426)}{1.35} = \frac{252}{1.35} = 187 \text{ kips}$$

$$R_m = \frac{2nM_P}{h_c} = \frac{2(3)3.64S}{14.5} = 187, \therefore S = 124 \text{ in.}^3$$

Try 12 WF 92 to satisfy buckling criteria (par. 4-04d).

$S = 125.0$ in.^{3}, $Z = 130.0$ in.^{3}, $I = 788.9$ in.^{4}, $0.5(S +$

$$a/t_w = 10.91/0.545 = 20 < 22$$

$$b/t_f = 12.155/0.856 = 14.2 \approx 14.0; \text{ OK}$$

$$M_P = 0.5(S + Z)f_{dy} = \frac{127.5(41.6)}{12} = 441 \text{ kip-ft}$$

$$R_m = \frac{2nM_P}{h_c} = \frac{2(3)441}{14.5} = 182 \text{ kips}$$

$$k = \frac{12EIn}{h^3} = \frac{12(30)10^3(788.9)3}{16^3(144)} = 1440 \text{ kips/ft}$$

$$x_e = R_m/k = 182/1440 = 0.126 \text{ ft}$$

$$x_m = \alpha\beta\, x_e = 12(0.126) = 1.52 \text{ ft}$$

$$T_n = 2\pi\sqrt{m/k} = 6.28\sqrt{2.64/1440} = 0.268 \text{ sec}$$

f. Second Trial - Work Done vs Energy Absorption Capacit

$$T/T_n = 0.081/0.268 = 0.302$$

$$C_R = R_m/B = 182/595 = 0.306$$

$t_m/T = 1.8$, $t_m = 1.8(0.081) = 0.146$ sec (fig. 5.29)

Revise the load-time curve as above (par. 5-13).

Impulse up to $t = 0.18$ sec in figure 7.11 is $H = 1.196$ ps

$$T = 2H/B = 2(1.196)/25.3 = 0.094 \text{ sec}$$

$$T/T_n = 0.094/0.268 = 0.351$$

$t_m/T = 1.8$, $t_m = 1.8(0.094) = 0.17$ sec; OK (fig. 5.29)

$C_W = 0.82$ (fig. 5.27)

$$W_P = \frac{H^2}{2m} = \left(\frac{BT}{2}\right)^2/2m = \frac{(595)^2 0.094^2}{8(2.64)} = 149.0 \text{ ft-kips}$$

$$W_m = C_W W_P = 0.82(149) = 122 \text{ ft-kips}$$

$$E = R_m\left(x_m - \frac{x_e}{2}\right) = 182\left(1.52 - \frac{0.126}{2}\right) = 266 \text{ ft-kips}$$

This column section is ample. Another trial may be made to reduce the size further.

g. Third Trial - Actual Properties.

$$R_m = \frac{0.5(W_m + E)}{y_m} = \frac{0.5(122 + 266)}{1.52} = 128 \text{ kips}$$

$$R_m = \frac{2nM_P}{h_c} = \frac{2(3)3.64(S)}{14.5} = 128, \therefore S = 85 \text{ in.}^3$$

Try 10 WF 77, $S = 86.1$ in.^{3}, $Z = 97.6$ in.^{3}, $0.5(S + Z) = 91.8$ in.^{3},
$I = 457.2$ in.^{4}, $A = 22.67$ in.^{2}

$a/t_w = 8.89/0.535 = 16.6 < 22$; OK

$b/t_f = 10.195/0.868 = 11.75 < 14.0$; OK

$$M_P = 0.5(S + Z)f_{dy} = \frac{91.8(41.6)}{12} = 318 \text{ kip-ft}$$

$$R_m = \frac{2nM_P}{h_c} = \frac{2(3)318}{14.5} = 131 \text{ kips}$$

$$k = \frac{12EIn}{h^3} = \frac{12(30)10^3(457.2)3}{16^3(144)} = 835 \text{ kips/ft}$$

$x_e = R_m/k = 131/835 = 0.157$ ft

$x_m = \alpha\beta\, x_e = 12x_e = 12(0.157) = 1.88$ ft

$T_n = 2\pi\sqrt{m/k} = 6.28\sqrt{2.64/835} = 0.353$ sec

h. Third Trial - Work Done vs Energy Absorption Capacity.

$T/T_n = 0.094/0.353 = 0.27$

$C_R = R_m/B = 131/595 = 0.22$

$t_m/T = 2.5$, $t_m = 2.5(0.094) = 0.235$ sec (fig. 5.29)

Revise the load-time curve as above (par. 5-13).

Impulse up to time = 0.25 sec is 1.30 psi-sec (fig. 7.11)

$T = 2H/B = 2(1.30)/25.3 = 0.103$ sec

$T/T_n = 0.103/0.353 = 0.292$

$t_m/T = 2.5$ (fig. 5.29), $t_m = 2.5(0.103) = 0.257$ sec; OK

$C_W = 0.88$ (fig. 5.26)

$$W_P = \frac{H^2}{2m} = \left(\frac{BT}{2}\right)^2/2m = \frac{(595)^2(0.103)^2}{8(2.64)} = 179 \text{ ft-kips}$$

$$W_m = C_W W_P = 0.88(179) = 157 \text{ ft-kips}$$

$$E = R_m\left(x_m - \frac{x_e}{2}\right) = 131\left(1.88 - \frac{0.157}{2}\right) = 236 \text{ ft-kips}$$

This size is accepted for preliminary design. As noted abo desirable to have E exceed W because of the approximations in liminary design procedure.

i. Shear Stress Check of 10 WF 77.

$$\text{Estimated } V_{max} = \frac{R_m}{3} = \frac{(131)}{3} = 44 \text{ kips in one column}$$

$$v = \frac{V}{t_w d} = \frac{44{,}000}{0.535(10.62)} = 7700 \text{ psi}$$

Allowable v = 21,000 psi (par. 4-05c); OK

j. Slenderness Criterion for Beam Columns. (See par. 4-08 proximate evaluation of the column slenderness criterion is made column size is accepted for final analysis. The criterion is:

$$\left(\frac{M_D}{M_P}\right)\left(\frac{K'Ld}{100bt}\right) + \left(\frac{P_D}{P_P}\right)\left(\frac{K''L}{15r}\right) < 1.0 \text{ (eq 4.10)}$$

$$M_P = 318 \text{ kip-ft}$$

$$M_D = \frac{W_m}{E} M_P = \frac{157}{236}(318) = 212 \text{ kip-ft (a rough estimate)}$$

$$P_P = f_{dy}A = 41.6(22.67) = 945 \text{ kips}$$

$$P_D = \frac{(3.7)144(43.5)(18)}{(1000)3} = 139 \text{ kips, at } t_m = 0.257 \text{ sec (fig.}$$

$$K' = 0.14 \text{ (table 4.1)}$$

$$K'' = 0.50 \text{ (table 4.2)}$$

$$L = 14.5 \text{ ft (clear height)}$$

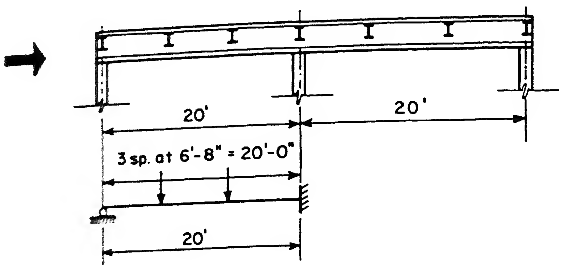
$r = 2.60$ in., $b = 10.195$ in., $t_f = 0.868$ in., $d = 10.62$ i

Substituting gives

$$\left(\frac{212}{318}\right)\left[\frac{0.14(14.5)12(10.62)}{100(10.195)0.868}\right] + \left(\frac{139}{945}\right)\left[\frac{0.5(14.5)12}{15(2.60)}\right]$$

$$= 0.196 + 0.329 = 0.53 < 1.0; \text{ OK}$$

7-25 DESIGN OF ROOF GIRDER. The girders are rolled structural steel shapes continuous over two spans. Since the procedures of this manual consider only single-span elements, the girders are designed for vertical loads as beams fixed at the interior support and pinned at the exterior support. The vertical loads are of two kinds; uniformly distributed load due to the girder dead weight and two concentrated loads, static plus blast applied by the purlins at the girder third-points.



The design moment at the interior support is a combination of three superposed bending moments as follows:

(1) The moment caused by the static loads;

(2) The moment caused by the vertical blast loads; and

(3) The moment imposed by the central column in restraining lateral motion of the frame.

The girder is designed for elastic behavior so that at all times it will be capable of providing the full restraint equal to the column plastic moment at each column support. For this two-span girder the column moment in the girder is equal to one-half the plastic hinge moment of the column (par. 7-11).

The basic design procedure is essentially the same as the elastic design procedure illustrated in paragraph 6-12. Although this is an elastic design, the limiting moment is the plastic resisting moment of the section (par. 4-04b).

The preliminary design load is based on an idealized version of the purlin dynamic reactions. After obtaining a satisfactory preliminary design the actual average purlin dynamic reaction is used in a numerical integration analysis to verify the preliminary design.

a. Loading. The critical girder loading results from the blast wave moving parallel to the girder axis. For this condition the average roof loads vary along the axis perpendicular to the girder from a maximum at the ends to a minimum over the central portion of the roof (par. 3-08d). To

obtain the vertical blast load on the girder it is necessary to d

(1) The variation of Zone 3 local overpressure;

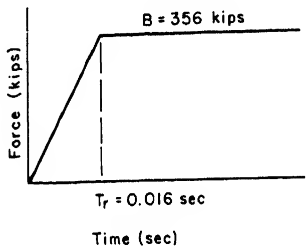
(2) The slab dynamic reactions for these overpressures; and

(3) The purlin dynamic reactions when loaded by the slab dy reactions.

All this has been done in paragraph 7-23 in designing the p The girder dynamic load is determined in figure 7.19 by adding th reactions of purlins (A) and (B) plotted with the proper lag time

$$t_{lag} = \frac{\text{purlin spacing}}{\text{velocity of blast wave}} = \frac{6.67}{1403} = 0.0048 \text{ sec}$$

The dynamic reactions are obtained from tables 7.6 and 7.7 (par. The design load as idealized from the computed loading shown by f is defined by:



$B = 4(89) = 356$ kips

$T_r = 0.016$ sec

b. Elastic Range Dynamic Design Fa

(Refer to table 6.1.)

Concentrated mass:

$K_L = 0.81,$ $\quad K_M = 0.67,$ $\quad K_{LM} = 0.83$

$R_m = 6M_p/L,$ $\quad k = \dfrac{132EI}{L^3}$

$V_1 = 0.17R + 0.17P,$ $\quad V_2 = 0.33R + 0.33P$

Uniform mass:

$K_L = 0.81,$ $\quad K_M = 0.45,$ $\quad K_{LM} - 0.83$

c. Mass Computation.

Slab and roofing $\left[\dfrac{3.75(150)}{12} + 6.0\right] \dfrac{18(43.5)}{(1000)2} = 20.7$ kips

Purlins $\dfrac{42.4(18)2}{1000} = 1.5$ kips

Girder (estimate) $\dfrac{160(20)}{1000} = 3.2$ kips

Connections (allow 10%) $= 0.1(3.2 + 1.5) = 0.47$ kips

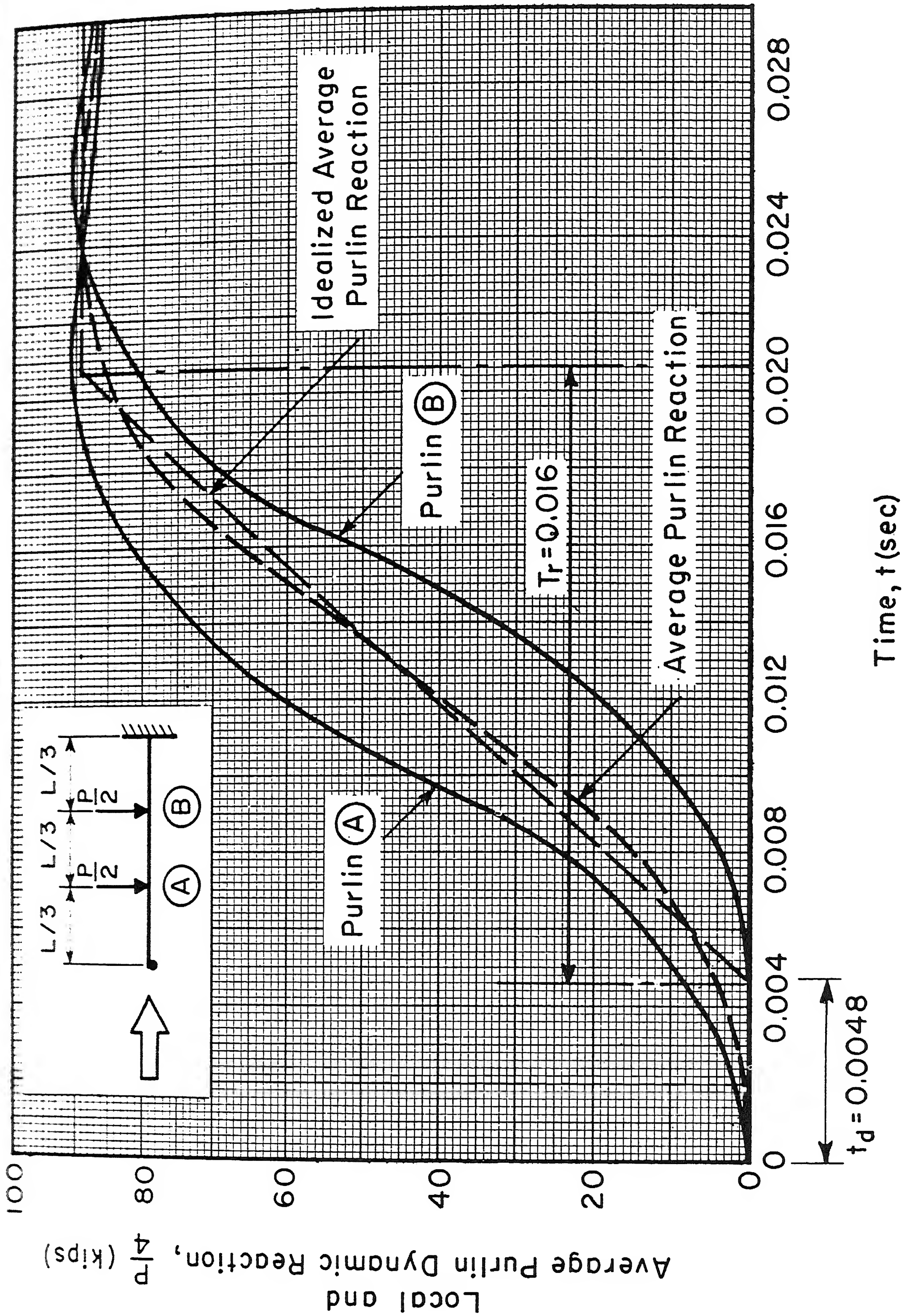


Figure 7.19. Determination of average girder load vs time curve

mass $m = \frac{(2/3)(20.7) + (1.5)}{32.2} =$

$\frac{\ldots + 0.47}{32.2} = 0.115$ kip-sec²

Properties.

(experience)

= 500 kips (par. 6-11)

= 3.64S kip-ft (S in in

∴ S = 458 in.³

.5(S + Z) = 542 in.³,

= 1880 kip-ft (eq 4.

30,800 kips/ft

Properties.

(.475) = 0.318 kip-s

) = 0.052 kip-sec²/ft

kip-sec²/ft

kips/ft

= 0.0241 sec

( ) = 516 kips

1720 kip-ft

kip-ft (pars. 7-12

support due to static lo

( )(.67)(6.67 + 20)0.25 +

= 9.25 + 22.7 + 28.3

for vertical blast loads

kip-ft < 1720 kip-ft

f. <u>Second Trial - Actual Properties.</u> Try 36 WF 160

$S = 541 \text{ in.}^3, \quad Z = 622 \text{ in.}^3, \quad 0.5(S + Z) = 581 \text{ in.}^3, \quad I = 9739 \text{ in.}^4$

$M_P = f_{dy}\ 0.5(S + Z) = 41.6(581) = 2020$ kip-ft

$$k = \frac{132EI}{L^3} = \frac{132(30)10^3(9739)}{20^3(144)} = 33{,}300 \text{ kips/ft}$$

g. <u>Second Trial - Equivalent Properties.</u> The change in the mass is neglected.

$k_e = K_L k = 0.81(33{,}300) = 27{,}000$ kips/ft

$T_n = 2\pi\sqrt{m_e/k_e} = 6.28\sqrt{0.37/27{,}000} = 0.0232$ sec

$T_r/T_n = 0.016/0.0232 = 0.69$

D.L.F. = 1.38 (fig. 5.21)

Required R_m = D.L.F.(B) = 1.38(356) = 491 kips

Required $M = R_m L/6 = 491(20)/6 = 1640$ kip-ft

Girder resistance available for vertical blast loads

$M_P - M_S - 0.5(M_P)_{column} = 2020 - 60 - 159 = 1800$ kip-ft; OK

$t_m/T_r = 1.19$ (fig. 5.21)

$t_m = 0.016(1.19) = 0.019$ sec; OK

h. <u>Preliminary Design for Shear Stress.</u>

At $t_m = 0.019$ sec, $t = 0.019 + 0.0048 = 0.0238$ sec

$P = 4(89) = 356$ kips (fig. 7.19)

$V = 0.33R_m + 0.33P = 0.33(491 + 356) = 280$ kips

$$v = \frac{V}{t_w d} = \frac{280{,}000}{0.653(36.0)} = 11{,}700 \text{ psi}$$

Allowable v = 21,000 psi (par. 4-05c)

i. <u>Determination of Maximum Deflection and Dynamic Reactions by Numerical Integration.</u>

$$R_m = \frac{6M_P}{L} = \frac{6(1800)}{20} = 540 \text{ kips}$$

$k = 33{,}300$ kips/ft

$$y_e = \frac{R_m}{k} = \frac{540}{33{,}300} = 0.0162 \text{ ft}$$

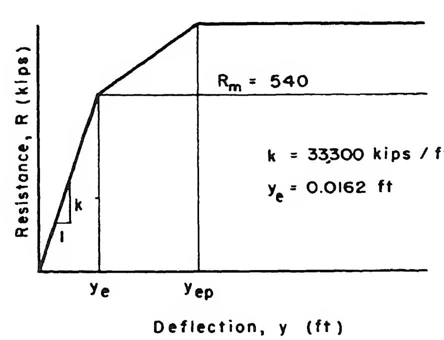


Figure 7.20. Resistance function for 36 WF 160 girder spanning 20 ft, fixed at one end and pinned at the other

Table 7.9. Determination of Maximum Deflection and Dynamic Reactions for

t (sec)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)
0	0	0	0.65	0.000006	0
0.002	4.0	0.2	3.8	0.000033	0.000006
0.004	11.2	1.5	9.7	0.000085	0.000045
0.006	30.0	5.6	24.4	0.000214	0.000169
0.008	56.0	17.0	39.0	0.000341	0.000507
0.010	110.0	39.0	71.0	0.000621	0.001186
0.012	160.0	83.0	77.0	0.000674	0.002486
0.014	208.0	149.0	59.0	0.000516	0.004460
0.016	266.0	231.0	35.0	0.000306	0.006950
0.018	312.0	325.0	-13.0	-0.000114	0.009746
0.020	337.0	414.0	-77.0	-0.000674	0.012428
0.022	351.0	481.0	-130.0	-0.001137	0.014436
0.024	356.0	510.0	-154.0	-0.001347	0.015307
0.026	356.0	494.0			0.014831

The basic equation for the numerical integration in tabl

$$y_{n+1} = \ddot{y}_n(\Delta t)^2 + 2y_n - y_{n-1} \text{ (table 5.3)}$$

where

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(\Delta t)^2}{K_{LM}(m)} = \frac{(P_n - R_n)(0.002)^2}{(0.55)(0.115) + (0.83)(0.475)}$$

$$= 8.75 \times 10^{-6} (P_n - R_n) \text{ ft}$$

The time interval $\Delta t = 0.002$ sec is approximately equal to $T_n/$ sec (par. 5-08). The dynamic reaction equations are listed in 7-25b.

The P_n values for the second column in table 7.9 are obt figure 7.19 multiplying by 4 to obtain the total concentrated to the girder by the two purlins.

j. Shear Stress Check.

V = 286 kips (table 7.9)

$$v = \frac{V}{dt_w} = \frac{286,000}{36.0(0.653)} = 12,150 \text{ psi}$$

Allowable v = 21,000 psi; OK (par. 4-05c)

k. Check Proportions of 36 WF 160 for Local Buckling.

$b/t_f = 12.0/1.02 = 11.8 < 14$; OK

$a/t_w = 33.96/0.653 = 52 > 30$; NG, longitudinal stiffeners required (see par. 4-06d)

$t_s = 3/4 > 0.653$; OK

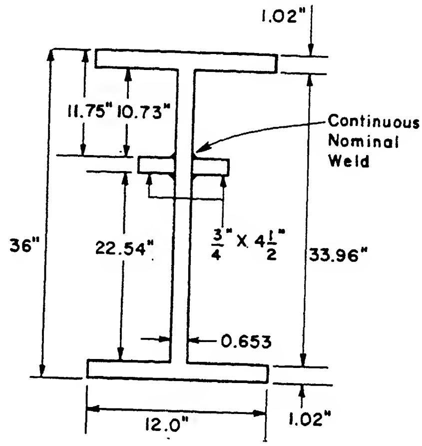
$b_s/t_s = 6$; $b_s = 6t_s = 6(3/4) = 4.5$ in.

Use longitudinal stiffeners, 2 plates 3/4 in. by 4-1/2 in. by full length.

$c/t_w = 10.73/0.653 = 16.5 < 22$; OK

$e/t_w = 22.54/0.653 = 34.5 < 40$; OK

Weight of added plate stiffeners equals 23 lb/ft.



7-26 FINAL DESIGN OF COLUMN. The column design was begun in paragraph 7-24. The final steps in the column design are illustrated by this paragraph. The steps which follow are preliminary to the numerical analysis to determine the lateral deflection of the top of the columns. In the preliminary design (par. 7-24) some of the factors which affect the response are neglected to simplify the computations. In this paragraph these factors are considered: the variation of plastic hinge moment with direct stress, the variation of column resistance with lateral deflection, the effect of girder flexibility on the stiffness of the columns, and the difference between the load on the wall slab and the dynamic reactions from the wall which are used as the lateral design load for the frame columns.

a. Mass Computation. (Refer to par. 7-24b.)

Walls = 86.5 kips, roof slab = 41.4 kips, purlins = 5.3 kips

$$\text{Girder} = \frac{160(41)}{1000} = 6.5 \text{ kips}$$

$$\text{Columns} = \frac{77(3)14.5}{1000} = 3.35 \text{ kips}$$

Connections = 1.2 kips

Mass of single-degree-of-freedom system = total roof + 1/3 (columns and walls)

$$m = \frac{41.4 + 5.3 + 6.5 + 0.33(86.5 + 3.35)}{32.2} = \frac{83.2}{32.2} = 2.6 \text{ kip-sec}^2\text{/ft}$$

b. Column Properties.

10 W 77

$I = 457.2 \text{ in.}^4$, $S = 86.1 \text{ in.}^3$, $Z = 97.6 \text{ in.}^3$, $0.5(S + Z)$

$A = 22.67 \text{ in.}^2$, $b = 10.195$ in., $t_f = 0.868$ in.

$a = 8.89$ in., $t_w = 0.535$ in., $d = 10.62$ in.

c. Column Interaction Design Data. The plastic hinge m axial load, and the values of M_1 and P_1 are computed below fro properties.

$$M_P = f_{dy}(S + Z)0.5 = \frac{41.6}{12}(91.8) = 318 \text{ kip-ft (eq 4.2)}$$

$$P_P = f_{dy}A = 41.6(22.67) = 945 \text{ kips (eq 4.7)}$$

$$P_1 = \frac{f_{dy}}{d}\left[2bt_f^{\,2} + \frac{t_w d^2}{2} - 2t_w t_f^{\,2}\right] \quad \text{(eq 4.12)}$$

$$P_1 = \frac{41.6}{10.62}\left[2(10.195)(0.868)^2 + \frac{0.535(10.62)^2}{2} - 2(0.535)(\right.$$

$$P_1 = 3.91\,[15.4 + 30.2 - 0.80] = 175 \text{ kips}$$

$$M_1 = \frac{f_{dy}}{3d}\left[4t_w\left(\frac{d}{2} - t_f\right)^3 + bt_f\left(3d^2 - 6dt_f + 4t_f^{\,2}\right)\right] \quad \text{(eq 4.}$$

$$= \frac{41.6}{(12)3(10.62)}\left\{4(0.535)\left(\frac{10.62}{2} - 0.868\right)^3 + \right.$$

$$10.195(0.868)\,[3(10.62)^2 - 6(10.62)0.868 + 4(0.868)^2$$

$$M_1 = \frac{1.30}{12}\left\{187 + 8.85\,[339 - 55.5 + 3.0]\right\} = \frac{1.3}{12}(2727)$$

For $P_D > P_1$

$$M_D = \left(\frac{P_P - P_D}{P_P - P_1}\right)M_1 = \frac{945 - P_D}{945 - 175}(296) = 364 - 0.384P_D \quad \text{(eq}$$

For $P_D < P_1$

$$M_D = M_P - \frac{P_D}{P_1}(M_P - M_1) \quad \text{(eq 4.14)}$$

$$= 318 - \frac{(318 - 296)P_D}{175} = 318 - 0.125\,P_D$$

d. Effect of Girder Flexibility. The relative flexibili girders reduces the spring constant k in the elastic range fr

obtained in paragraph 7-24 for infinitely stiff girders (par. 7-08). To obtain this revised value of k a simple sidesway analysis of the frame is made. From the sidesway analysis the magnitude of the lateral force required to cause a unit displacement is determined.

In figure 7.22 the initial slope, which is the proper spring constant until formation of the first hinge, is assumed to hold up to the plastic resistance R_m. The true resistance curve is represented approximately by the dashed line up to R_m.

For infinitely rigid girders k = 835 kips/ft (par. 7-24g).

The elastic sidesway analysis in figure 7.21 is started with -1000 kip-ft at top and bottom of each column. This is equivalent to a lateral displacement of the frame,

$$X = (\text{F.E.M.})h^2/6EI$$

$$= \frac{(1000)(16)^2 144}{6(30)10^3(457.2)}$$

$$= 0.448 \text{ ft}$$

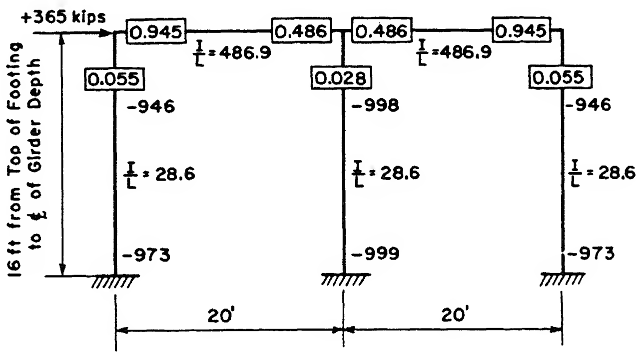


Figure 7.21. Sidesway frame analysis by moment distribution

In the elastic sidesway analysis the conventional procedure of using centerline dimensions is adopted although in all other computations the clear height of the column is used.

$$R = \frac{\Sigma M}{h} = \frac{2(946 + 973) + (999 + 998)}{16}$$

$$= 365 \text{ kips}$$

$$k = \frac{R}{x} = \frac{365}{0.448} = 815 \text{ kips/ft}$$

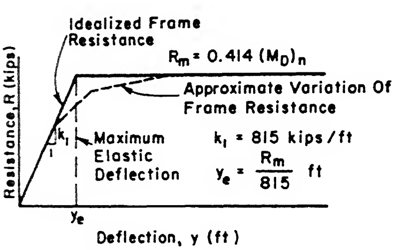
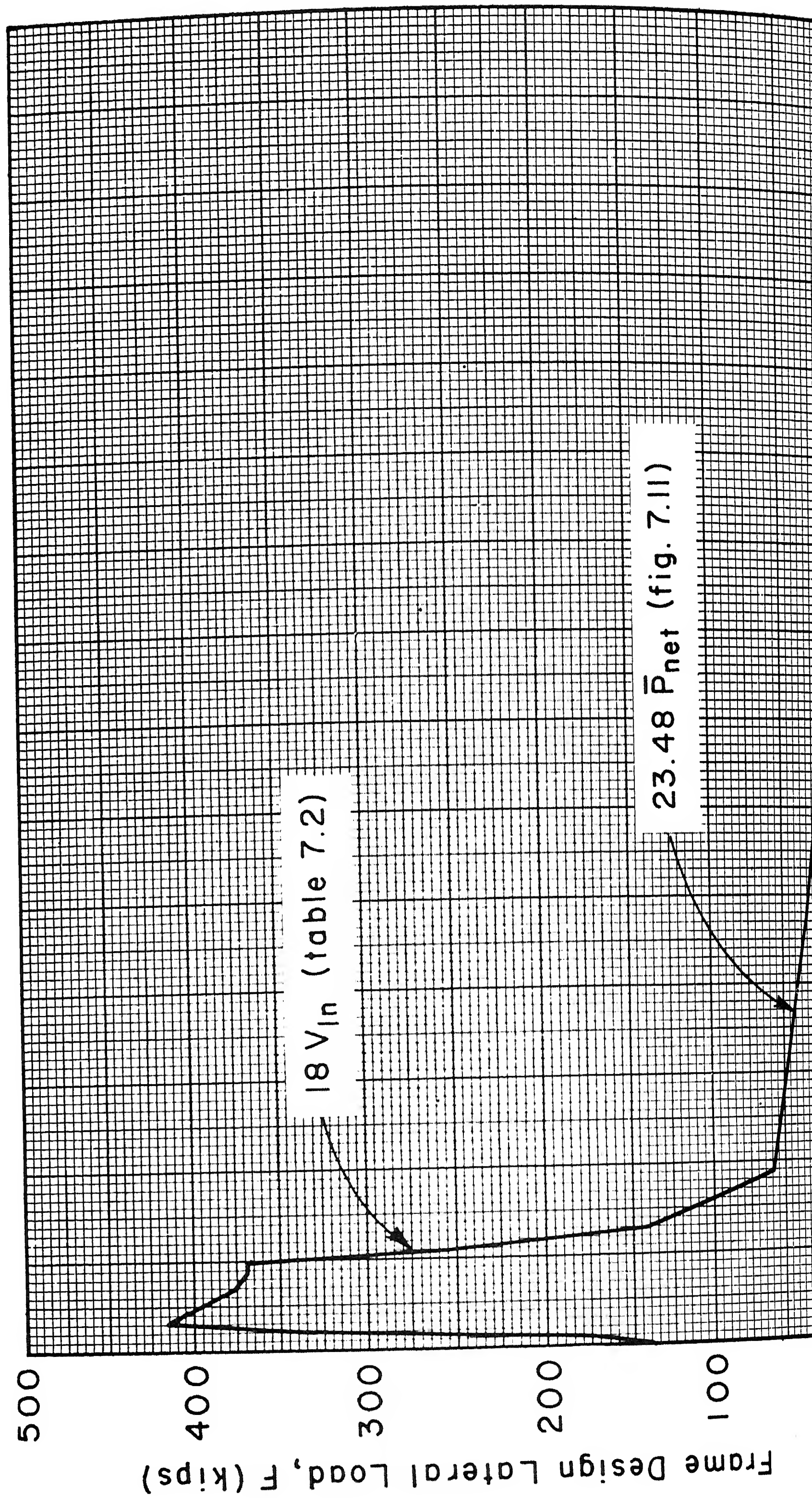


Figure 7.22. Resistance diagram for 10 WF 77 columns

e. Loading. Both horizontal and vertical loads are considered. The lateral load for the numerical integration F_n (fig. 7.23) is obtained from the V_{1h} dynamic reaction column of the numerical integration analysis of the front wall slabs in paragraph 7-21g (table 7.2). The dynamic reaction values for a one-foot width are multiplied by the width of one bay. After


18 V_{In} (table 7.2)
23.48 $\bar{P}_{net}$ (fig. 7.11)
500
400
300
200
100
Frame Design Lateral Load, F (kips)

the dynamic reaction of the front wall slab has decreased to the level of the applied load (at t = 0.055 sec) the F_n values are determined from the net lateral load curve (fig. 7.11) where $F_n = [144(18)9.06]/1000\ \overline{P}_{net} = 23.48\ \overline{P}_{net}$ kips ($\overline{P}_{net}$ is in psi). These data are plotted in figure 7.23 to give the frame lateral design load for use in table 7.10.

The effective height of the loaded area is obtained from one-half the wall clear height plus the thickness of the roof slab,

$$h = \frac{17.5}{2} + \frac{3.75}{12} = 9.06 \text{ ft}$$

f. Computation of Deflection of Frame by Numerical Integration. (Refer to table 7.10.) The total vertical load P_n in the second column is obtained by multiplying the average roof overpressure (fig. 7.12) by $[43.5(18)144]/1000 = 112.8$ and adding the dead weight of the roof system. The $(P_D)_n$ values are the average axial column loads and are obtained by dividing the total vertical load by the number of columns. The $(P_D)_n$ column is used in the plastic range to obtain the value of $(M_D)_n$ from the interaction design equations (par. 7-26c). The value of $(M_D)_n$ is used to obtain the maximum resistance at any time by the relation $(R_m)_n = [2n(M_D)_n]/h_c = [2(3)(M_D)_n]/14.5 = 0.414(M_D)_n$.

R_n is equal to $kx_n = 815x_n$ in the elastic range. In the plastic range R_m is the limiting value of R. The expression $(P_n x_n)/h_c$ indicates the decrease in resistance corresponding to the increase in moment resulting from the eccentric loading. The basic equation for the numerical integration in table 7.10 is

$$x_{n+1} = \ddot{x}_n(\Delta t)^2 + 2x_n - x_{n-1} \quad \text{(table 5.3)}$$

where

$$\ddot{x}_n(\Delta t)^2 = \frac{\left[F_n - R_n + \frac{P_n}{h_c}(x_n)\right](\Delta t)^2}{m}$$

$$\ddot{x}_n(\Delta t)^2 = \frac{(0.02)^2\left[F_n - R_n + \frac{P_n}{h_c}(x_n)\right]}{2.6} = 1.54 \times 10^{-4}\left[F_n - R_n + \frac{P_n}{h_c}(x_n)\right]$$

To check the slenderness criteria (par. 4-08) the following equation is evaluated for each time interval.

Table 7.10. Determination of Column Adequacy

t (sec)	P_n (kips)	$(P_D)_n$ (kips)	$(M_D)_n$ (kip-ft)	$(R_m)_n$ (kips)	$\frac{P_n}{h_c}$ (kip-ft)	F_n (kips)	R_n (kips)	$\frac{P_n x_n}{h_c}$ (kips)	$F_n - R_n + \frac{P_n x_n}{h_c}$ (kips)	$\ddot{x}_n(\Delta t)^2$ (ft)	x_n (ft)	$0.292 \frac{(M_D)_n}{M_P}$	2.23
0	55	18	316	131	3.8	139.0	0	0	69.5	0.0106	0	0.29	
0.02	698	233	275	114	48.1	405.0	8.6	0.5	396.9	0.0607	0.0106	0.25	
0.04	998	333	236	98	68.8	371.0	66.7	5.6	309.9	0.0474	0.0819	0.217	
0.06	913	304	247	102	63.0	207.0	102.0	12.0	117.0	0.0179	0.2006	0.23	
0.08	836	279	257	106	57.7	150.0	106.0	19.0	13.0	0.0020	0.3372	0.24	
0.10	769	256	266	110	53.0	58.0	110.0	25.0	-27.0	-0.0041	0.4758	0.24	
0.12	711	237	273	113	49.0	54.0	113.0	30.0	-29.0	-0.0044	0.6103	0.25	
0.14	663	221	279	116	45.7	50.0	116.0	34.0	-32.0	-0.0049	0.7404	0.26	
0.16	621	207	285	118	42.8	46.0	118.0	37.0	-35.0	-0.0054	0.8656	0.26	
0.18	584	195	289	120	40.3	42.0	120.0	40.0	-38.0	-0.0058	0.9854	0.27	
0.20	551	184	293	121	38.0	38.0	121.0	42.0	-41.0	-0.0063	1.0994	0.27	
0.22	523	174	296	123	36.1	34.0	123.0	44.0	-45.0	-0.0069	1.2071	0.27	
0.24	496	165	297	123	34.2	30.0	123.0	45.0	-48.0	-0.0073	1.3079	0.27	
0.26	471	157	298	123	32.5	27.0	123.0	46.0	-50.0	-0.0076	1.4014	0.27	
0.28	447	149	299	124	30.8	24.0	124.0	46.0	-54.0	-0.0083	1.4873	0.27	
0.30	424	141	300	124	29.2	22.0	124.0	46.0	-56.0	-0.0086	1.5649	0.28	
0.32	401	134	301	125	27.7	19.0	125.0	45.0	-61.0	-0.0093	1.6339	0.28	
0.34	380	127	302	125	26.2	16.0	125.0	44.0	-65.0	-0.0099	1.6936	0.28	
0.36	359	120	303	125	24.8	14.0	125.0	43.0	-68.0	-0.0104	1.7434	0.28	
0.38	337	112	304	126	23.2	12.0	126.0	41.0	-73.0	-0.0112	1.7828	0.28	
0.40	318	106	305	126	21.9	11.0	126.0	40.0	-75.0	-0.0115	1.8110	0.28	
0.42	297	99	306	127	20.5	9.5	127.0	37.0	-80.0	-0.0122	1.8277	0.28	
0.44	278	92	307	127	19.2	8.0	127.0	35.0	-84.0	-0.0128	1.8323	0.28	
0.46											1.8241		

$$\frac{M_D}{M_P}\left[\frac{K'Ld}{100bt_f}\right] + \frac{P_D}{P_P}\left[\frac{K''L}{15r}\right] \leqslant 1 \text{ (eq 4.10)}$$

$K' = 0.14$, $K'' = 0.50$, $L = h_c = 14.5$ ft $= 174$ in. (tables

$$\frac{M_D}{M_P}\left[\frac{0.14(174)10.62}{100(10.195)(0.868)}\right] + \frac{P_D}{P_u}\left[\frac{0.5(174)}{15(2.60)}\right] \leqslant 1$$

$$0.292\frac{M_D}{M_P} + 2.231\frac{P_D}{P_P} \leqslant 1$$

The time interval Δt used in table 7.10 is based on the natura

$T_n = 2\pi\sqrt{m/k} = 6.28\sqrt{2.6/815} = 0.353.$

$$t = 0.02 < \frac{T_n}{10} = 0.035 \text{ (par. 5-08)}$$

The allowable maximum displacement $= \frac{\alpha\beta\ R_m}{k} = \frac{12(102)}{815} = 1.5$ ft and 7-24c). The computations in table 7.10 show that the slen terion is satisfied because the combined ratio is only very sl 1.0 (0.3%). The computed maximum displacement = 1.56 ft, thus

$$\alpha\beta = \frac{1.56}{1.5}(12) = 12.5; \text{ OK.}$$

7-27 FOUNDATION DESIGN. a. General. The foundation of this building is designed by both of the methods described in paragraph 6-31 primarily to illustrate the procedures. The first design is according to the Quasi-Static Foundation Design Procedure in paragraph 6-31c, and the second is according to the Dynamic Foundation Design Procedure in paragraph 6-31d. A preliminary plan of one bay of the foundation is presented by figure 7-24.

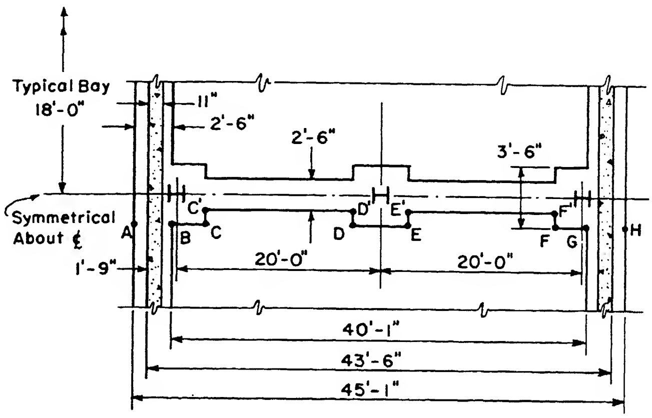


Figure 7.24. Preliminary foundation plan for one bay

b. Loads. The first requirement of the quasi-static analysis is a tabulation of all the lateral and vertical loads on the foundation. Because each numerical integration performed previously is based on a separate value of Δt, it is necessary to plot curves of the variation of each of the forces with time. The locations of the forces on the foundation are shown by the sketch on the left. The vertical loads include the column and footing blast loads and the dead load of the entire structure.

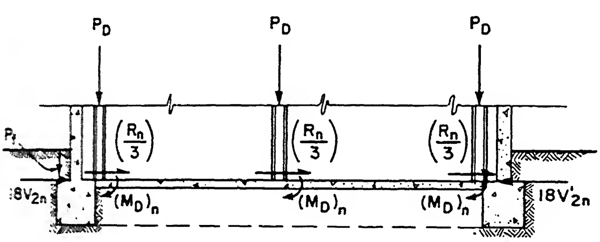


Figure 7.25 is a curve showing the time variation of the total column (blast plus static) load $P_n = 3(P_D)_n$ obtained from data in table 7.10. Figure 7.26 is a curve of the time variation of the blast load on the projecting front footing. The front wall footing is estimated at 2 ft 6 in. for the preliminary analysis so that the projecting area is $\left(2.5 - \frac{11}{12}\right)0.5 = 0.79$ ft. The vertical blast load on the footing is obtained by multiplying the projecting area by the front face overpressure (fig. 7.9) thus obtaining

$$0.79(18)\overline{P}_{front}\ (144/1000) = 2.05\overline{P}_{front}$$

Figure 7.27 is the total vertical load curve obtained by ad
ordinates of figures 7.25 and 7.26. The blast load on the rear f
neglected herein.

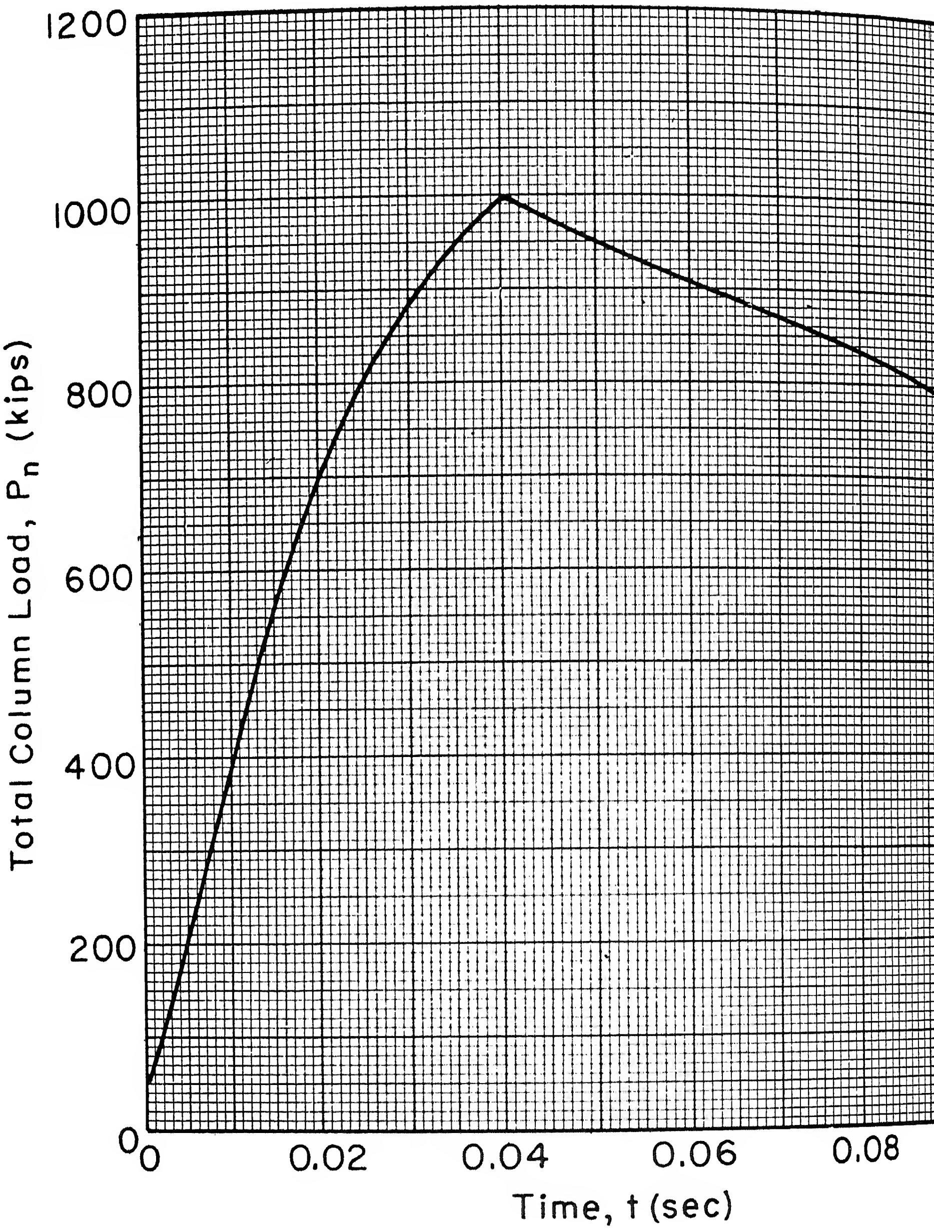


Figure 7.25. Total column load vs time

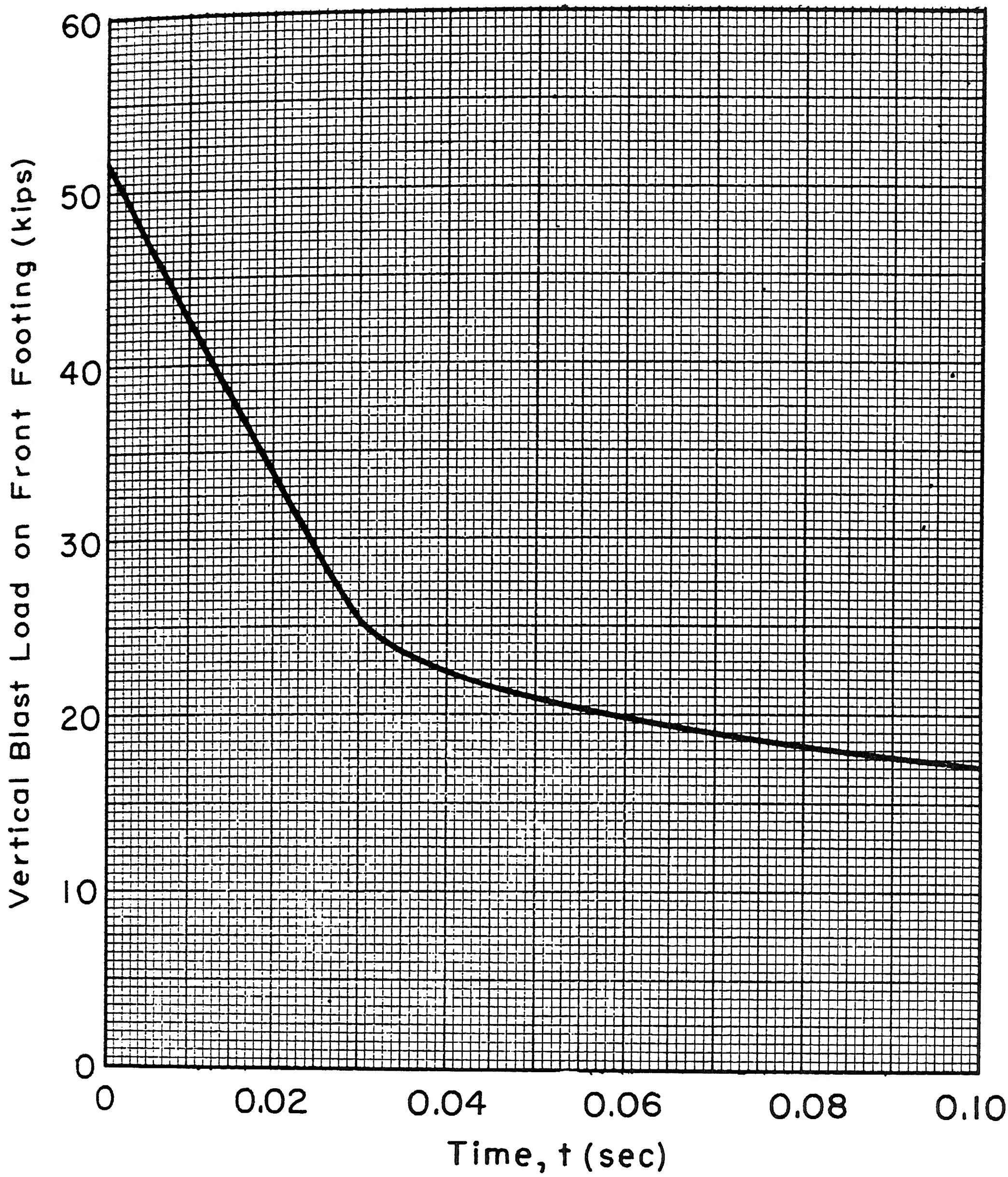


Figure 7.26. Vertical load on front wall footing projection vs time

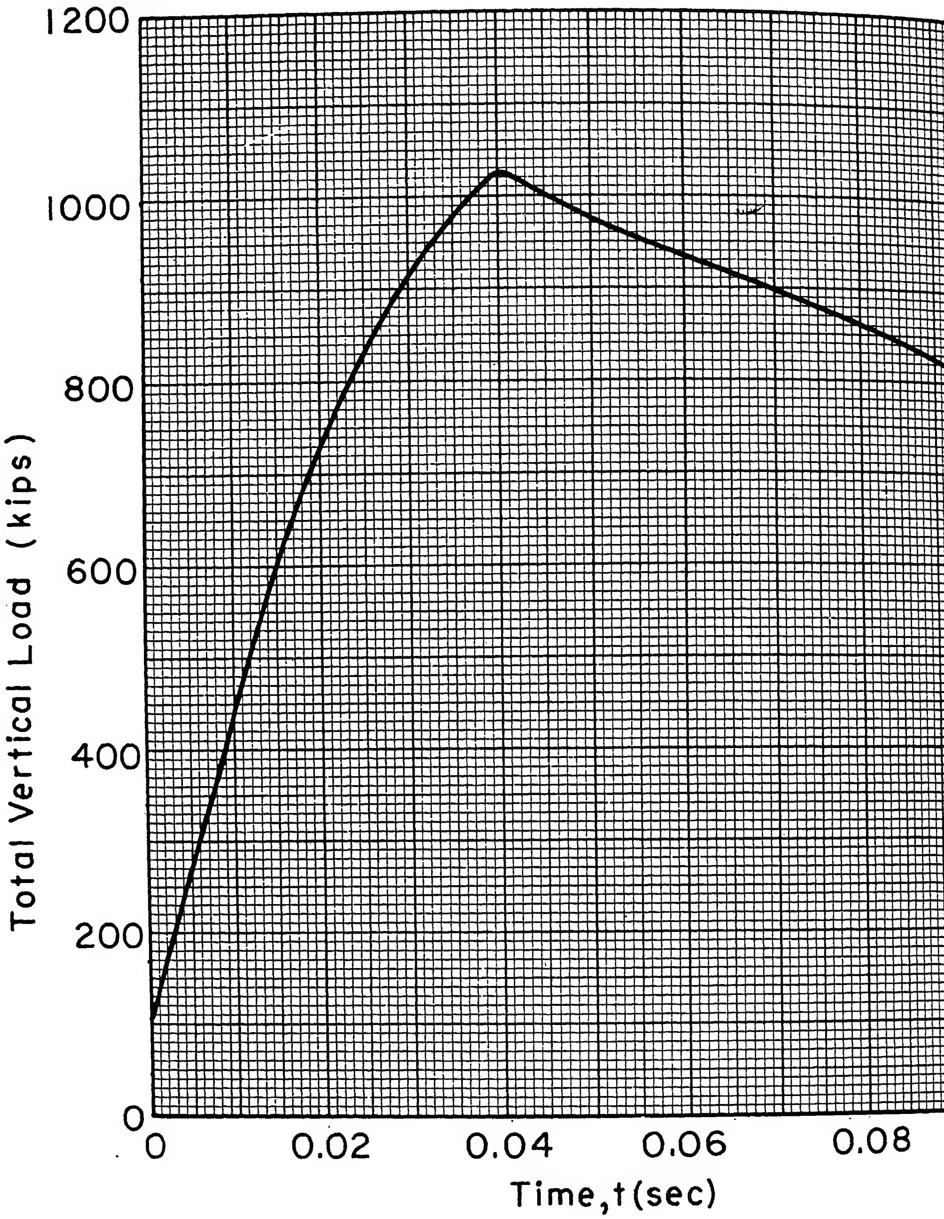


Figure 7.27. Total vertical load vs time

The lateral loads include the shear in the columns and the front and rear wall slab reactions at the wall footings. The rear wall slab reactions are obtained in table 7.11 by a numerical integration using the rear wall overpressure data in figure 7.10 and the following data from paragraph 7-21g.

$P_n = 2.52\ \overline{P}_{back}$

$y_e = 0.0344$ ft

$y_{ep} = 0.090$ ft

$k_1 = 793$ kips/ft

$k_{ep} = 329$ kips/ft

$R_m = 45.6$ kips

$\Delta t = 0.005$ sec

$\ddot{y}_n(\Delta t)^2 = 4.29(10^{-4})(P_n - R_n)$ ft, elastic range

$\ddot{y}_n(\Delta t)^2 = 4.29(10^{-4})(P_n - R_n)$ ft, elasto-plastic range

$\ddot{y}_n(\Delta t)^2 = 5.07(10^{-4})(P_n - R_n)$ ft, elasto-plastic range

$\ddot{y}_n(\Delta t)^2 = 5.07(10^{-4})(P_n - R_n)$ ft, plastic range

Table 7.11. Determination of Dynamic Reactions for Back Wall Slab

t (sec)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}(\Delta t)^2$	y_n (ft)	V_{1n} (kips)	V_{2n} (kips)
0.030	0.216	0	0.216	0.000093	0	0.0	0.0
0.035	1.210	0.073	1.137	0.000488	0.000093	0.2	0.3
0.040	2.520	0.534	1.986	0.000852	0.000674	0.4	0.7
0.045	3.654	1.671	1.983	0.000851	0.002107	0.9	1.4
0.050	4.788	3.482	1.306	0.000560	0.004391	1.5	2.4
0.055	6.048	5.737	0.311	0.000133	0.007235	2.2	3.6
0.060	7.207	8.098	-0.891	-0.000382	0.010212	3.0	4.9
0.065	8.392	10.156	-1.764	-0.000757	0.012807	3.6	6.0
0.070	9.576	11.613	-2.037	-0.000874	0.014645	4.2	6.8
0.075	10.786	12.378	-1.592	-0.000683	0.015609	4.5	7.4
0.080	12.096	12.601	-0.505	-0.000217	0.015890	4.7	7.7
0.085	13.306	12.652	+0.654	+0.000281	0.015954	4.9	8.0
0.090	14.440	12.925	+1.515	+0.000650	0.016299	5.1	8.3
0.095	15.624	13.714	+1.910	+0.000819	0.017294	5.4	8.9
0.100	15.498	15.153	+0.345	+0.000148	0.019108	5.8	9.5

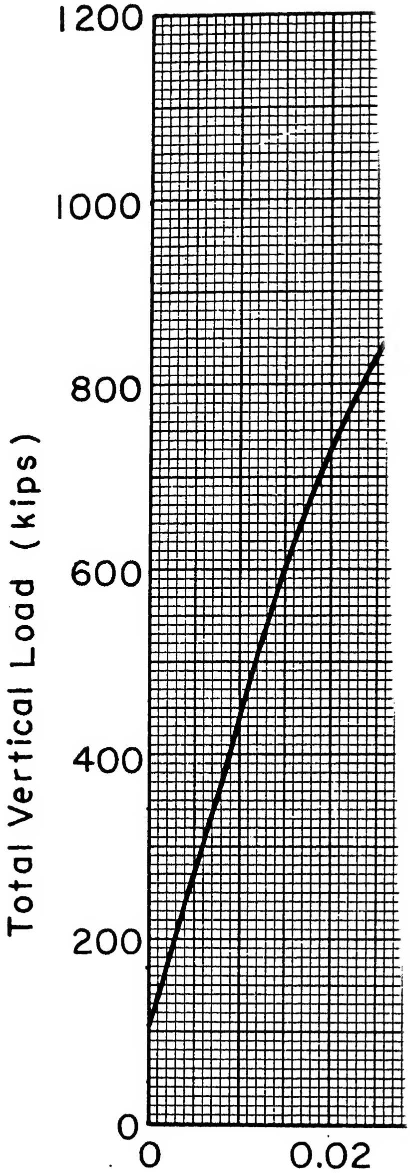


Figure 7.2

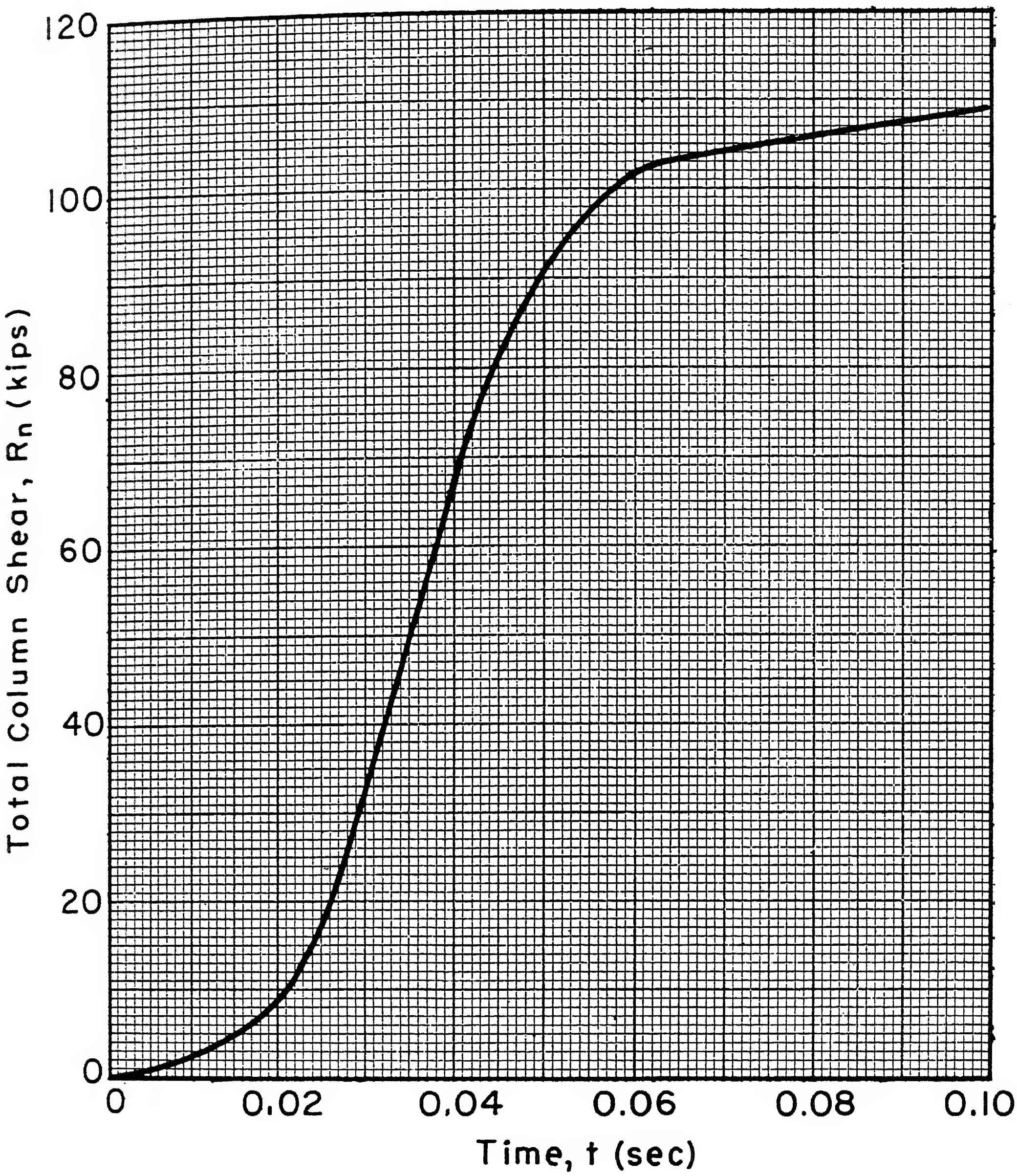


Figure 7.29. Column shear vs time

Figure 7.30 indicates the total lateral load-time variation on the foundation and is obtained by adding the ordinate values of figures 7.28 and 7.29.

Figure 7.28 is a plot of $18(V_{2n} - V'_{2n})$ where V_{2n} and V'_{2n} are, resp
the front and rear wall slab footing reactions from tables 7.2 and
After t = 0.07 sec, the curve is obtained from the net lateral loa
(fig. 7.11):

$$\frac{144(18)17.5}{1000(2)} \bar{P}_{net} = 22.6\bar{P}_{net} \text{ kips } (\bar{P}_{net} \text{ is in psi})$$

where 17.5 is the wall clear height.

The time variation of the column shear is plotted in figure
using data from table 7.10.

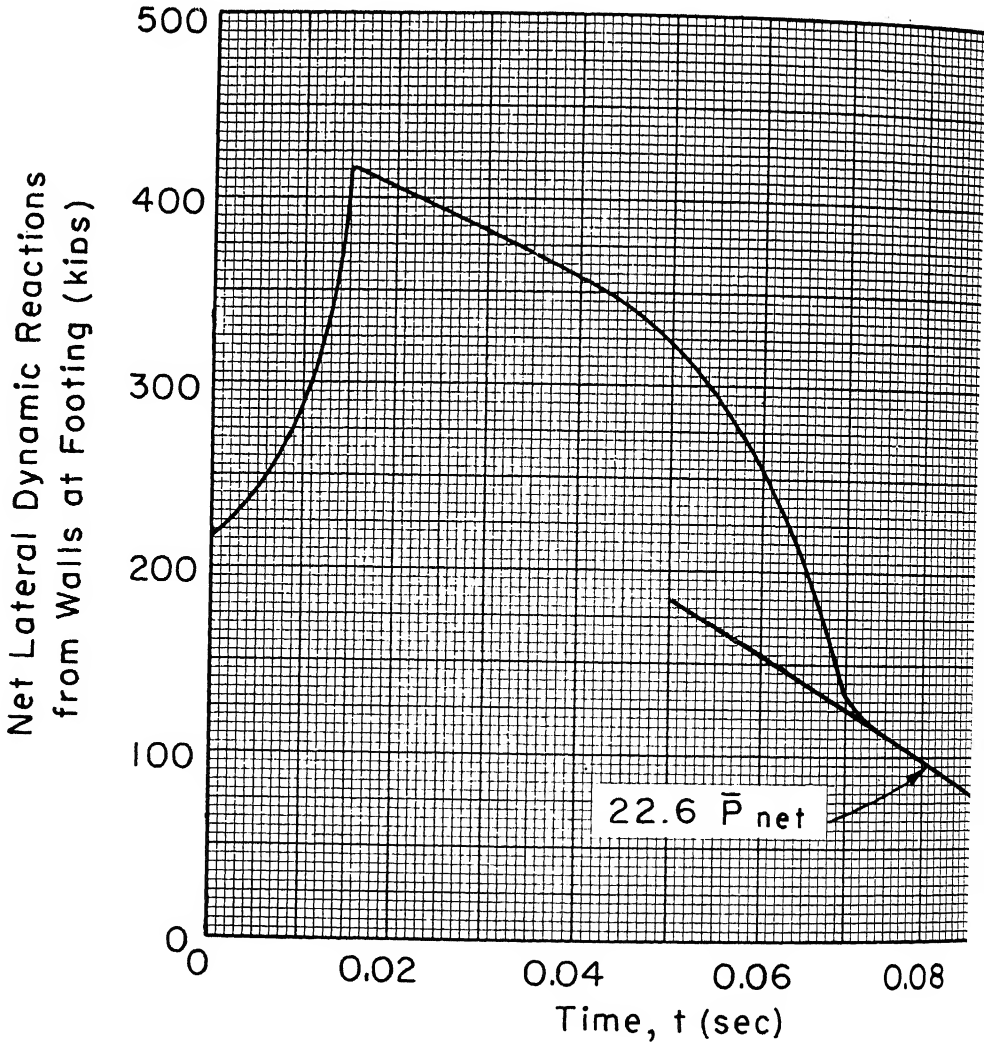


Figure 7.28. Wall reactions at foundation vs time

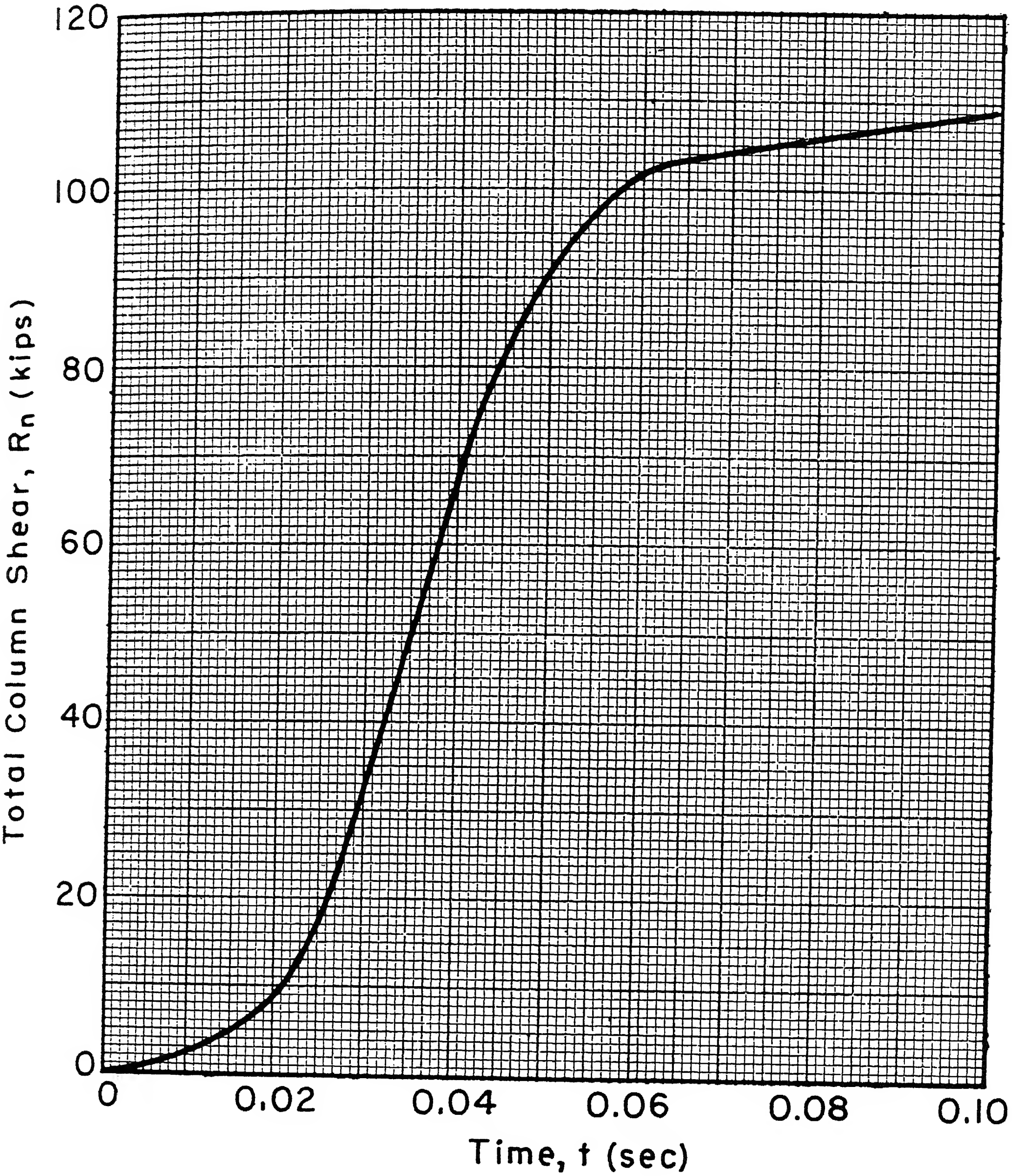


Figure 7.29. Column shear vs time

Figure 7.30 indicates the total lateral load-time variation on the foundation and is obtained by adding the ordinate values of figures 7.28 and 7.29.

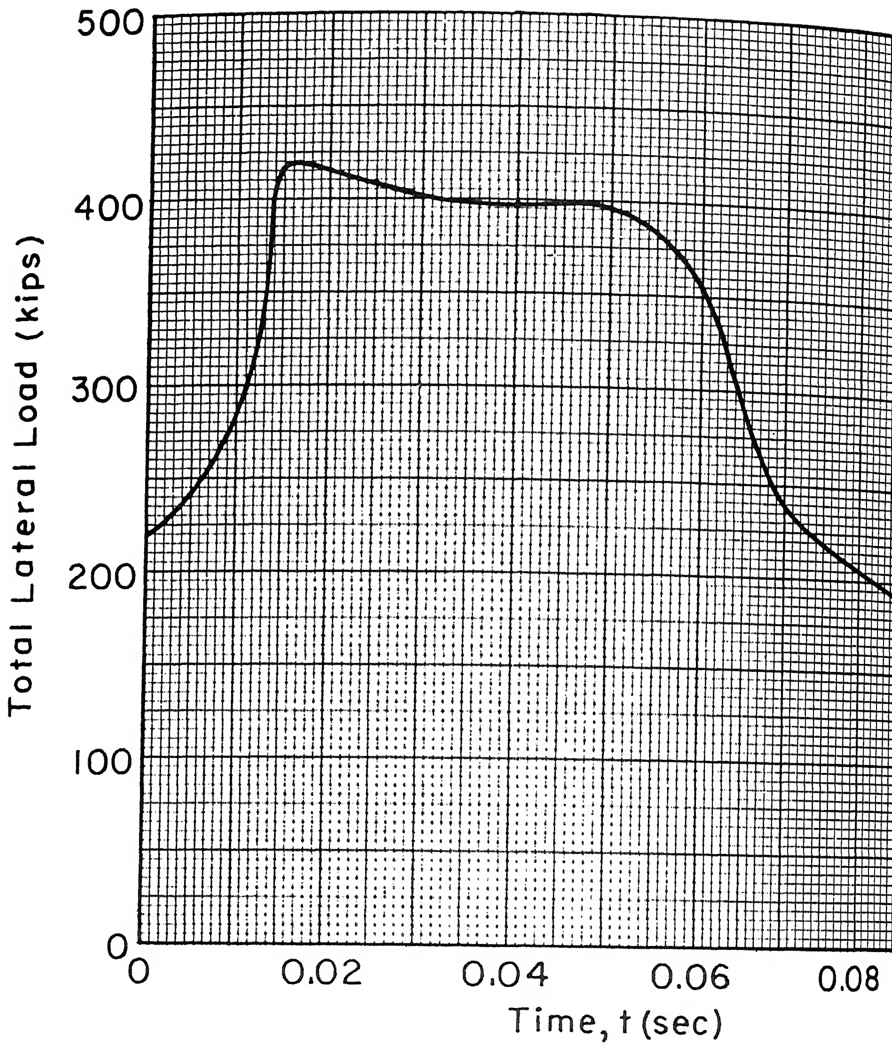


Figure 7.30. Total lateral load on foundation vs time

Although not needed until a later step, the time variatio column base moments is plotted in figure 7.31 using data obtai table 7.10. From t = 0 to t = 0.08 sec the columns are elasti the moment from one column on the foundation is

$$M = \frac{1}{3}\frac{h}{2}(R_n) = 2.42R_n$$

From t = 0.08 to 0.20 sec, the columns are plastic, hence $(M_D)_1$

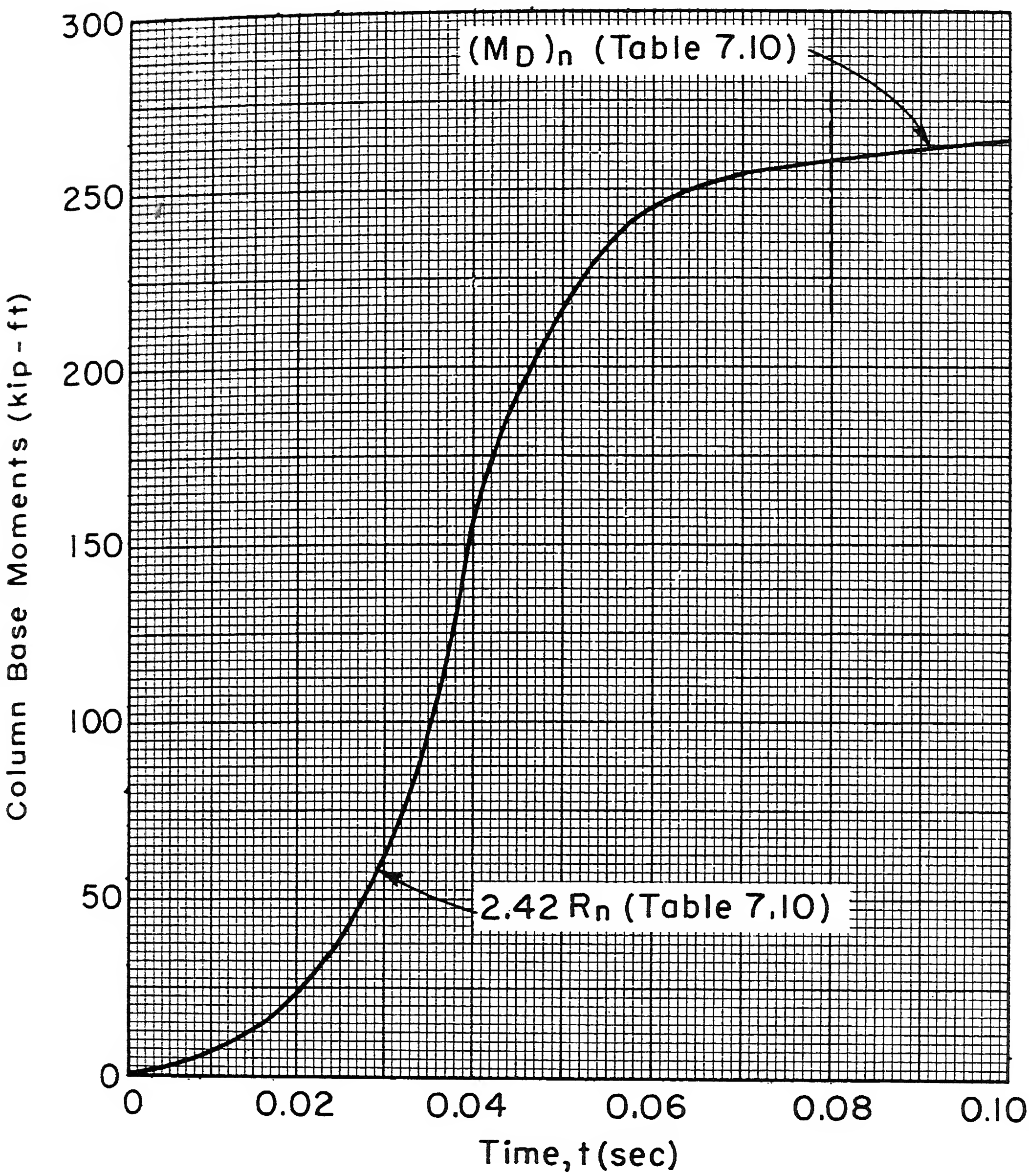


Figure 7.31. Column base moments vs time

c. Preliminary Sizes of Members. The preliminary column footing sizes are determined on the basis of the maximum column vertical load (fig. 7.25) and the dynamic design bearing capacity, 30 ksf

$$\text{Required area} = \frac{(P_D)_{max}}{30} = \frac{1000}{3(30)} = 11.1 \text{ sq ft}$$

Try column footings, 3 ft 6 in. square, area = 12.25 sq ft.

The size of the wall footing is determined by using the ma of blast load on the footing projection and adding to it the dea wall and overburden.

Dead load of exterior wall and footing: (assume wall heig footing width and depth = 2.5 ft, neglect overburden)

Wall = (2)(150/1000)(11/12)18(18.0) = 89.0 kips

Footing = (2)(150/1000)(2.5)2.5(18.0) = 33.8 kips

Total wall dead load = 89.0 + 33.8 = 122.8 kips

Maximum value of blast load = 54 kips (fig. 7.26)

$$\text{Theoretical width} = \frac{\text{total exterior wall load}}{\text{length (allowable bearing capacity)}}$$

$$\text{Width} = \frac{(122.8 + 54)}{2(18)30} = 0.164 \text{ ft}$$

This value is too small for practical purposes, hence an ar width of 2 ft 6 in. will be used for subsequent analyses.

d. <u>Preliminary Depth of Foundation.</u> The time variation of balanced load is obtained in table 7.12 by subtracting the availa

Table 7.12. Determination of Unbalanced Lateral Load

①	②	③	④	⑤	⑥
Time (sec)	Lateral Load (kips)	Vertical Load (kips)	Total Vertical Load (kips)	Friction (kips)	Unbalanced Load (kips
	fig. 7.30	fig. 7.27	③ + 163.6	0.5 ④	② -
0	218	105	268	134	84
0.005	243	270	433	217	26
0.010	288	440	603	302	None
0.015	420	600	763	382	
0.020	414	730	893	447	
0.030	403	905	1068	534	
0.040	397	1020	1183	592	
0.050	397	971	1134	567	
0.060	355	930	1093	547	

frictional force from the total lateral load. The lateral load causing sliding is obtained from figure 7.30. The frictional resistance to sliding is obtained by multiplying the total vertical load by the coefficient of friction, $\mu = 0.50$ (par. 7-18). The total vertical load at any time is obtained by adding the dead load of the total structure to the vertical loads in figure 7.27.

Columns = 3.35 kips (par. 7-23)

Column strap footings = 2.5(2.5)40(0.15) = 37.5 kips

Total dead load = 122.8 + 3.35 + 37.5 = 163.6 kips

The unbalanced lateral load is then the total lateral load minus the frictional resistance to sliding. The maximum value of this unbalance is 84 kips.

The procedure recommended in paragraph 6-31c requires that the depth of footing be that necessary to develop sufficient passive pressure resistance to equal twice the unbalanced lateral load at any time. The effect of the blast pressures acting on the surface of the ground at the back of the building should be included in determining the passive pressure resistance. In this example the back wall loading does not begin until 0.030 sec (table 7.11) and there is no unbalanced lateral load after t = 0.005 sec (table 7.12).

The required depth of footing is determined by:

$$F_P = K_{P\phi} \frac{\gamma H^2}{2} \quad \text{(eq 4.58)}$$

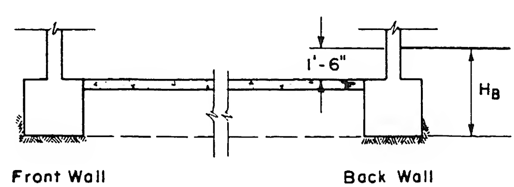
where

F_P = normal component of total passive resistance

= 2(maximum unbalance) = 2(84) = 168 kips

The resistance to lateral motion is provided by the lesser of: (1) combined passive pressure resistances of front and back footings or (2) frictional resistance of earth between footings plus passive pressure resistance of back footing.

Considering case (1) first, the length of back wall capable of resisting passive pressure is 18 ft. The effective height of soil is H_B.



The length of front wall capable of resisting passive pressur
(18 - 2.5) = 15.5 ft. The effective height of soil is (H_B) - (1.5
(increase for height of concrete floor) = $H_B - 1.5 + \frac{(150 - 100)}{100}$ (0
= H_B - 1.25 ft.

F_P = front wall resistance + back wall resistance

$$F_P = 15.5K_{P\phi}\frac{\gamma(H_B - 1.25)^2}{2} + 18K_{P\phi}\frac{\gamma(H_B)^2}{2}$$

$$168 = 15.5(10)0.100\frac{(H_B - 1.25)^2}{2} + 18(10)\frac{0.100(H_B)^2}{2}$$

H_B = 3.68 ft (compare available H_B = 4.31)

On this basis, the back footing is capable of providing a pas
sistance equal to

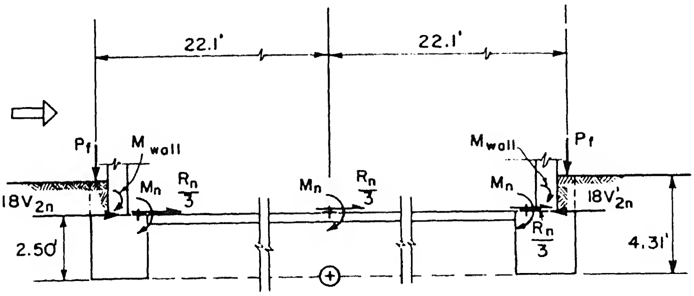
$$18K_{P\phi}\frac{\gamma(H_B)^2}{2} = \frac{18(10)0.100(3.68)^2}{2} = 121 \text{ kips}$$

The potential frictional resistance of soil between footings equals

$\mu W = \mu(\gamma)15.5(H_B - 1.25)41.7 = 0.50(0.100)15.5(2.43)41.7 = 78$

78 + 121 = 199 > 168, case (2) is OK.

e. Overturning Moment. The sketch below indicates the posit
loads considered in
overturning computat



In table 7.13,
time variation of th
turning moment on th
ture is determined.
wall reaction overtu
moment is obtained by multiplying the net wall reaction from figure
by 2.50 ft.

The wall support moments M_{wall} are obtained from data in tabl
and 7.11. In the elastic range $M_{wall} = (R_nL)/8$, for one bay M_{wall}
$= [(18)/8](17.5)R_n = 39.3R_n$. In the elasto-plastic and plastic ran
M_{wall} = 18(59.5) = 1070 kip-ft. The column base moments are obtain
multiplying the column moments (fig. 7.31) by the number of columns

Table 7.13. Determination of Overturning Moment

①	②	③	④	⑤	⑥	⑦	⑧	⑨	⑩	⑪
[illegible]	Net Column and Wall Reaction (kips)	Wall Reaction Overturning Moment (ft-kips)	Front Wall Support Moment (ft-kips)	Back Wall Support Moment (ft-kips)	Column Base Moments (ft-kips)	M_o (ft-kips)	Blast Load on Front Footing Projection (kips)	Blast Load on Back Footing Projection (kips)	Net Footing Projection Moments (ft-kips)	Net Overturning Moment (ft-kips)
Ref	fig. 7.30	② 2.50	table 7.2	table 7.11	fig. 7.31	[③ - ⑥]	fig. 7.26	fig. 7.32	22.1 [⑧ + ⑨]	[⑦ - ⑩]
[illegible]	218	545	0	0	0	545	52	0	1175	-630
[illegible]	240	600	425	0	4	1029	48	0	1085	-56
[illegible]	285	712	1070	0	15	1797	43	0	971	826
[illegible]	420	1050	1070	0	30	2150	39	0	883	1267
[illegible]	415	1038	1070	0	60	2168	34	0	770	1398
[illegible]	410	1025	1070	0	120	2215	30	0	680	1535
[illegible]	400	1000	1070	0	187	2257	26	0	589	1668
[illegible]	400	1000	1070	2.6	300	2373	24	1.0	520	1853
[illegible]	395	988	1070	21.0	480	2559	23	2.1	472	2087
[illegible]	395	988	1070	65.0	585	2708	22	2.9	431	2277
[illegible]	395	988	1070	136.0	651	2845	21	3.9	386	2459
[illegible]	380	950	944	225.0	705	2824	21	4.9	362	2462
[illegible]	355	888	583	317.0	741	2529			362	2167

bay. The footing projection righting moment is obtained by multiplying the blast load on front footing projection from figure 7.26 by 22.1 ft. The back footing overturning moment is obtained by multiplying the blast load on back footing projection from figure 7.32 by 22.1. The vertical blast load on the rear footing (fig. 7.32) is obtained by multiplying the projecting footing area by the rear face overpressure (fig. 7.10) obtaining

$$0.79(18)\overline{P}_{back}\frac{144}{1000} = 2.05\overline{P}_{back}$$

The overturning moment due to the passive pressure is neglected conservatively.

f. <u>Combined Axial Load and Overturning Moment.</u> From the equation for combined stresses it is possible to write

$$\text{Ultimate bearing capacity} = \frac{P}{A} + \frac{Mc}{I}$$

From which by substitution is obtained

$$30 = \frac{P}{197.5} + \frac{M(22.54)}{55{,}290}$$

$$P = 5929 - 0.0806M$$

where

Ultimate bearing capacity = 30 $kips/ft^2$ (par. 7-18)

P = total vertical load on one bay of foundation

A = total area of one bay of foundation = 197.5 ft^2

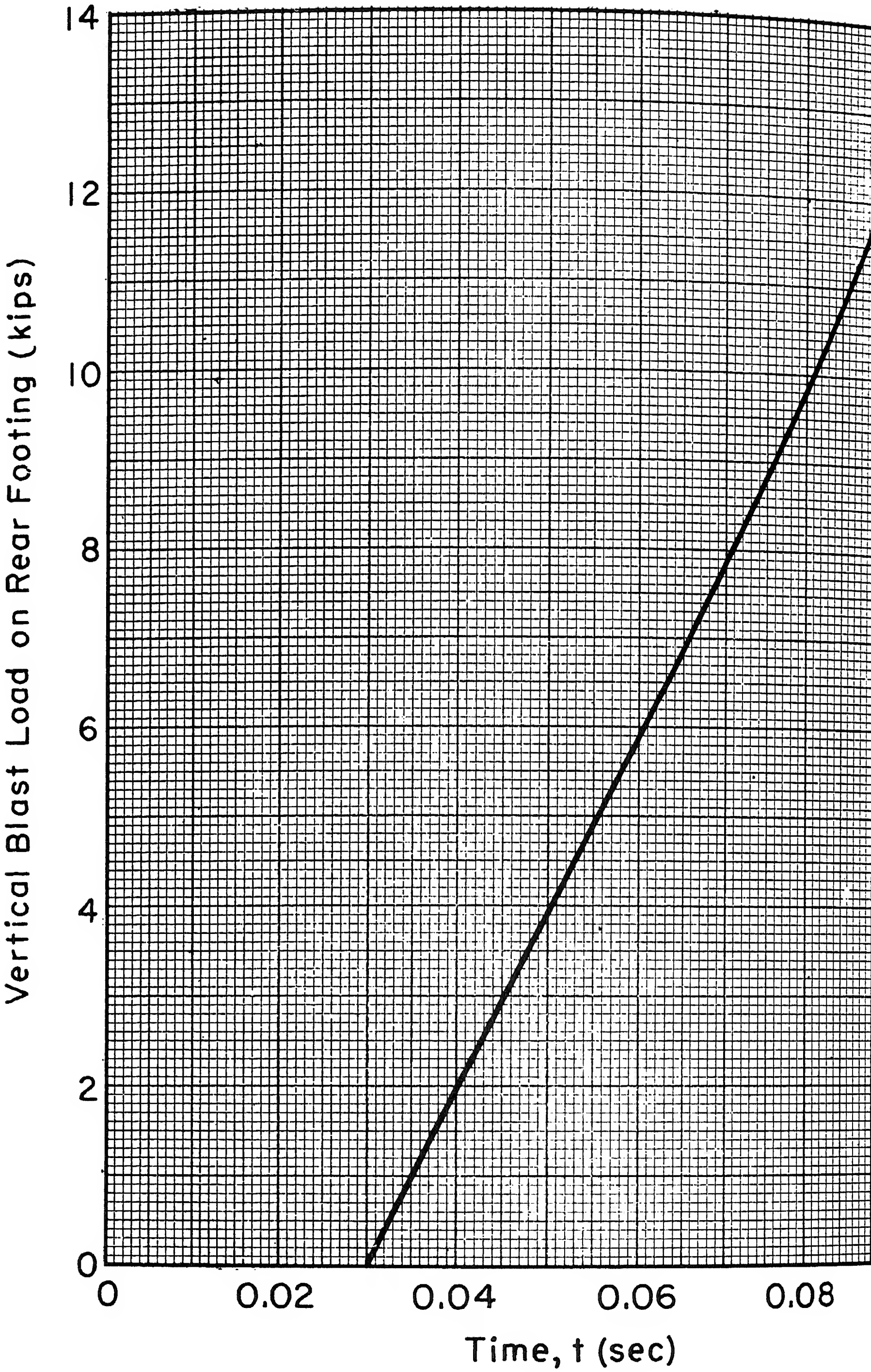


Figure 7.32. Vertical blast load on rear footing

M = total overturning moment on foundation

c = distance to extreme point on foundation from center

I = moment of inertia of foundation = 55,290 ft^4

Introducing the P and M data from table 7.13 into the equation for P (above) shows that the soil pressures and the preliminary sizes of the foundation elements are satisfactory.

g. Design of Foundation Members. In a complete design study it would be necessary to determine stresses at various times in order to determine the critical condition. To illustrate the procedure it is sufficient to compute stresses at only one time.

The column strap footing is designed for the stress conditions at t = 0.055 sec. One bay of the foundation 18 ft wide centered on a column strap footing is considered (fig. 7.33).

$$\frac{P}{A} = \frac{1113}{197.5} = 5.63 \text{ ksf}$$

(tables 7.12 and 7.13)

$$\frac{Mc}{I} = \frac{2462(22.54)}{55,290} = 1.00 \text{ ksf}$$

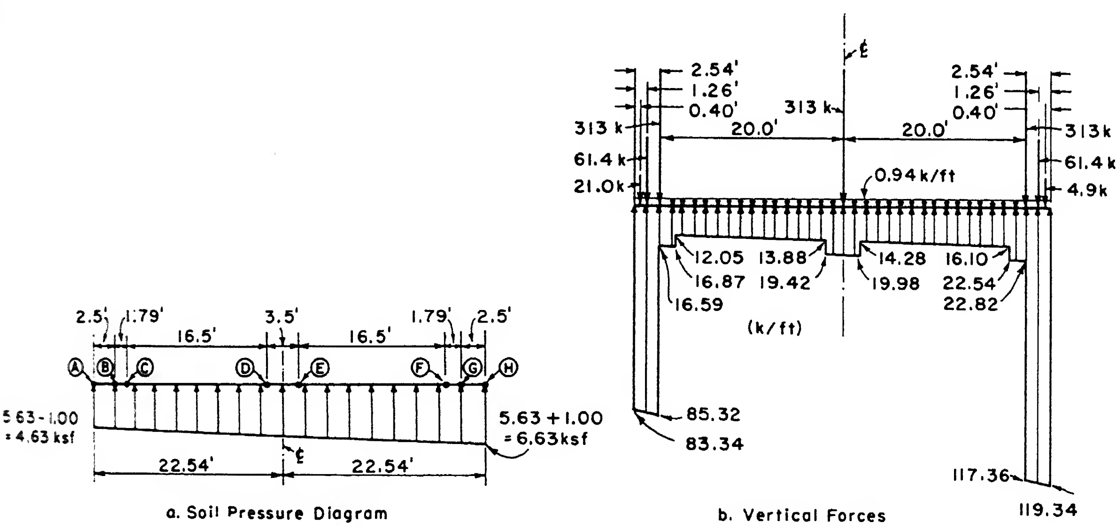
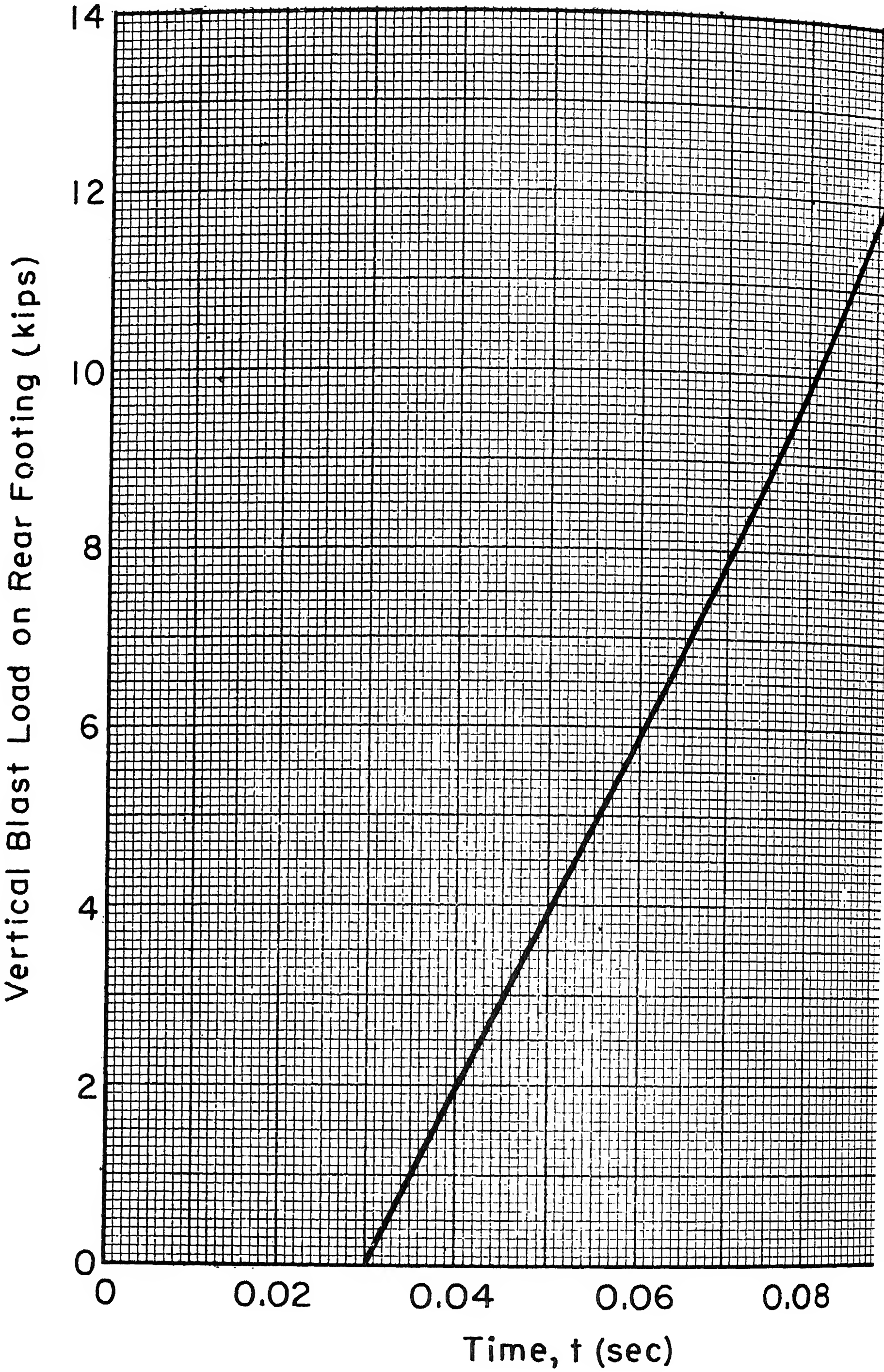


Figure 7.33. Vertical forces and soil pressure on foundation

The soil pressure variation is shown in figure 7.33a. The soil pressure multiplied by the footing width gives the soil load per foot (fig. 7.33b).



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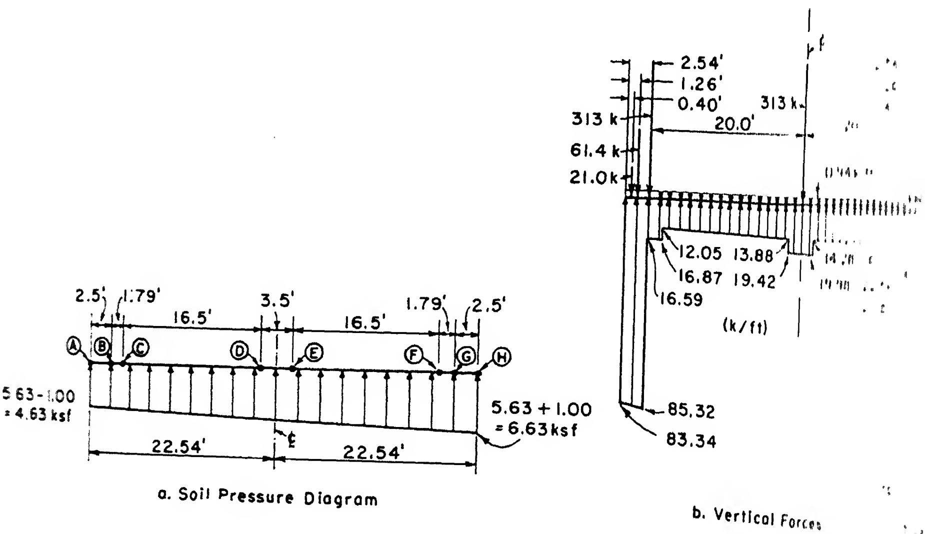


19.54) -

25)

Figure 7.33. Vertical forces and soil pressure on foundation

lly as-
bending
aced by
ied

The soil pressure variation is shown in figure 7.33a. The soil ... multiplied by the footing width gives the soil load per foot (fig. ...

In figures 7.33 and 7.34 all the forces and moments acti
foundation at time t = 0.055 sec are shown. They are as follo

Front wall = rear wall = 122.8/2 = 61.4 kips (par. 7-27c

Column strap footing = 37.5/40 = 0.94 kip-ft (par. 7-27c

Front footing blast load = 21.0 kips (table 7.13)

Rear footing blast load = 4.9 kips (fig. 7.32)

Column load (3.35 + 936)/3 = 313 kips (table 7.10)(fig.

Moment at base of front wall = 944 kip-ft (table 7.13)

Moment at base of rear wall = 225 kip-ft (table 7.13)

Shear at base of front wall = $18V_{2n}$ = 18(22.9) = 413 kip

Friction at base of footing = 557 kips (table 7.12)

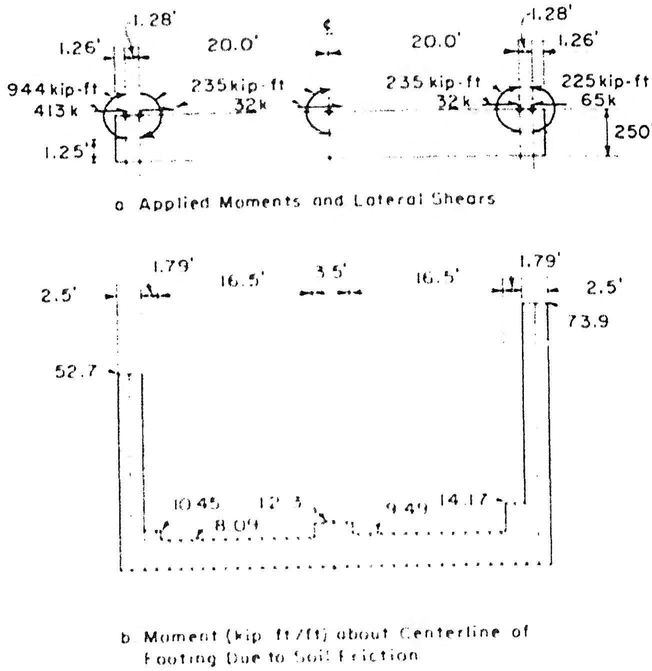


Figure 7.34. Applied moments and shears

The moments a
the front and rear
sumed to be transm
column strap footi
conservative proce
strap footing sinc
at the base of the
provided continuou
wall.

The friction
sumed to be distri
the base proportio
total vertical soi
example between po

$$W = \frac{83.34 + \ldots}{2}$$

= 210.8 ki

Friction = [illegible] (557) = [illegible]5.4 kips (table 7.12)

The critical sections are considered below.

<u>Shear Point E</u>

Σ loads down - Σ loads up (fig. 7.33b)

$$\frac{V}{1.5} = 4.9 + 61.4 + 313 + 18.29(0.94) - \left(\frac{119.34 + 117.3\ldots}{2}\right.$$

$$\left(\frac{[illegible]}{2}\right)1.79 + \left(\frac{16.10 + 14.28}{2}\right)16.5$$

$$\frac{V}{1.5} = 396.5 - 587.1 = -190.6 \text{ kips}$$

$$V = -190.6(1.5) = -286 \text{ kips}$$

Shear Point G

$$\frac{V}{1.5} = (4.9 + 614) - \left(\frac{117.36 + 119.34}{2}\right) 2.5$$

$$\frac{V}{1.5} = 66.3 - 295.9 = -229.6 \text{ kips}$$

$$V = 1.5(-229.6) = -344 \text{ kips}$$

Moment Point C (figs. 7.33 and 7.34)

$$\frac{M}{1.5} = 944 + 413(1.25) + 235 + 32(1.25) + 52.7(2.5) + \left(\frac{83.34 + 85.32}{2}\right) 2.5 \left(\frac{2.5}{2}\right) -21(2.14) - 61.4(1.28)$$

$$\frac{M}{1.5} = 2007$$

$$M = 3010 \text{ kip-ft}$$

Moment Point E (figs. 7.33 and 7.34)

$$\frac{M}{1.5} = -225 - 65(1.25) + 235 + 32(1.25) + 73.9(2.5) + 14.17(1.79) + 9.49(16.5) + 4.9(20.39) + 61.4(19.53) + 313(18.25) + 0.94(18.29)(9.14) - \left(\frac{117.36 + 119.34}{2}\right)(2.5)(19.54) - \left(\frac{22.82 + 22.54}{2}\right)(1.79)(17.39) - \left(\frac{14.28 + 16.10}{2}\right)(16.5)(8.25)$$

$$\frac{M}{1.5} = 1051$$

$$M = 1577 \text{ kip-ft}$$

The footing strap is checked to determine if the sizes originally assumed can be reinforced sufficiently. The sections are checked for bending moment capacity by equation 4.16 with the values of f_{dy} and f'_{dc} replaced by f_y and f'_c, respectively, because the loads are considered to be applied statically.

Section at Point E - b = 2.5 ft, assume d = 30 in.

$$M_p = pbdf_y d\left(1 - \frac{pf_y}{1.7f'_c}\right) \quad \text{(eq 4.16)}$$

For p = 0.02

$$M_p = 0.02(2.5)30(40)30\left[1 - \frac{0.02(40)}{1.7(3.0)}\right] = 1520 \text{ kip-ft}$$

M_p = 1520 kip-ft ≈ 1577 kip-ft; OK

Shear at this location = 286 kips

Shear stress $= v = \frac{8V}{7bd}$ (eq 4.23)

$$v = \frac{8(286)1000}{7(30)30} = 363 \text{ psi}$$

For no web reinforcement, allowable $v = 0.04f'_c + 5000p$ (eq 4.2

$= 0.04(3000) + (5000)0.02$

$= 120 + 100 = 220$ psi

Stirrups are supplied to provide (363 - 220 = 143 psi) additior quired shear strength.

$rf_y = 143$ (eq 4.24)

$$r = \frac{143}{40,000} = 0.0036$$

$$r = \frac{A_v}{bs} = \frac{A_v}{30(12)} = 0.0036$$

A_v = 1.29 sq in./ft

<u>Section at Point C</u> - b = 3.5 ft, assume d = 30 in.

$$M_p = pbdf_y d\left(1 - \frac{pf_y}{1.7f'_c}\right) \text{ (eq 4.16)}$$

For p = 0.03

$$M_p = 0.03(3.5)30(40)30\left[1 - \frac{0.03(40)}{1.7(3.0)}\right]$$

M_p = 2900 kip-ft ≈ 3010 kip-ft; OK

Maximum shear occurs at point G. This location has same cross tion as point C. For reversal of load on structure maximum shear wc cur at point C (V = 344 kips).

Shear stress $= v = \frac{8V}{7bd}$ (eq 4.23)

$$v = \frac{8(344)1000}{7(42)30} = 312 \text{ psi}$$

If no shear reinforcement is used, the allowable $v = 0.04f'_c$ + 5000p (eq 4.24b).

$v = 0.04(3000) + 5000(0.03) = 120 + 150 = 270$ psi

Shear reinforcement equal to (312 - 270 = 42 psi) is provided

$$r = \frac{37}{40,000} = 0.000925$$

A_v = rbs = 0.000925(42)12 = 0.47 sq in./ft

Since the critical sections are capable of resisting moments and shears, the entire foundation footing strap could be reinforced in proportion to moments and shears. The wall footings need only nominal reinforcement since dimensions selected are much larger than necessary for calculated soil pressure.

h. Dynamic Foundation Design. The dynamic design procedure (par. 6-31c) usually results in a more economical design than the quasi-static procedure because the dynamic effects are computed rather than conservatively estimated.

The results of the quasi-static design, with certain modifications illustrated in figure 7.35, will be used as a preliminary design in the dynamic design procedure. In this design the wall footings are placed eccentrically to assist in resisting overturning most effectively.

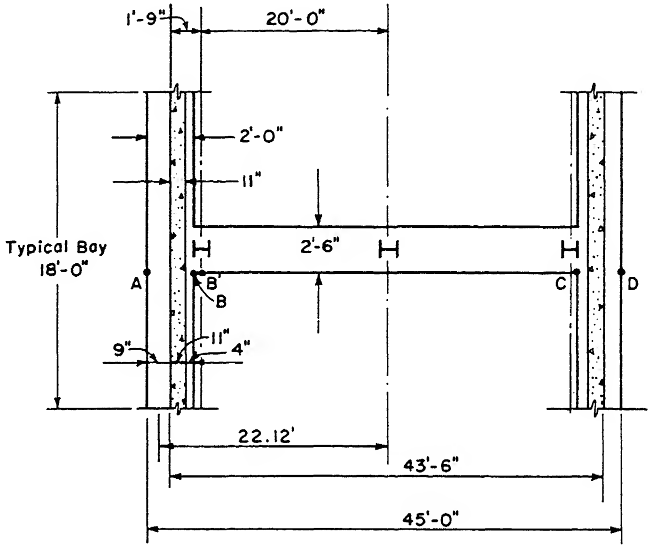


Figure 7.35. Plan of foundation

Using the principles presented in paragraph 6.31e for the sliding and overturning analysis, the horizontal acceleration $\ddot{x}_o$ and the angular acceleration α_o about the longitudinal axis through the bottom of the footings are determined. The corresponding horizontal displacement x_o and the angular displacement θ are determined as a function of time by means of a concurrent numerical integration of equations (6.94) and (6.95).

$$\alpha_o = \frac{M_o - F_o\bar{y}}{I_o - m\bar{y}^2} \quad \text{(eq 6.94)}$$

$$\ddot{x}_o = \frac{F_o}{m} - \alpha_o\bar{y} \quad \text{(eq 6.95)}$$

Element	Dimension	Volume (cu ft)	Weight (kips)	Mass (kip-sec^2/ft)	$\bar{y}$* (ft)	$m\bar{y}$ (ft)	d_1** (ft)	$\frac{md_1^2}{12}$ † (kip-sec^2)	$m\bar{y}^2$ (kip-sec^2)	I_{xx} (kip-sec^2)	d_2 †† (ft)	$\frac{md_2^2}{12}$ ‡ (kip-sec^2)	$\bar{x}$ (ft)	$m\bar{x}^2$ (kip-sec^2/ft)	I_{yy} (kip-sec^2/ft)
Roof slab	(See par. 7.21b)		41.4	1.285	20.32	26.11	(0.354)	-----	530.55	530.55	43.5	202.5			202.5
Purlin (A)	18 ft long		0.76	0.024	19.48	0.47	0.54	0.07	9.16	9.23	0.12	0	20.0	9.6	9.6
Purlin (B)	18 ft long		0.76	0.024	19.48	0.47	0.54	0.07	9.16	9.23	0.12	0	13.34	4.27	4.27
Purlin (C)	18 ft long		0.76	0.024	19.48	0.47	0.54	0.07	9.16	9.23	0.12	0	6.67	1.07	1.07
Purlin (D)	18 ft long		0.76	0.024	19.48	0.47	0.54	0.07	9.16	9.23	0.12	0			0
Purlin (E)	18 ft long		0.76	0.024	19.48	0.47	0.54	0.07	9.16	9.23	0.12	0	6.67	1.07	1.07
Purlin (F)	18 ft long		0.76	0.024	19.48	0.47	0.54	0.07	9.16	9.23	0.12	0	13.34	4.27	4.27
Purlin (G)	18 ft long		0.76	0.024	19.48	0.47	0.54	0.07	9.16	9.23	0.12	0	20.0	9.6	9.6
Girder	40 ft 10 in. long		7.50	0.233	18.65	4.35	1.19	0.33	101.40	101.77	0.20	0.01			0.01
Column 1	14.5 long		1.12	0.035	9.90	0.35	14.5	0.61	3.47	4.08	0.37	0	20.0	14.0	14.0
Column 2	14.5 long		1.12	0.035	9.90	0.35	14.5	0.61	3.47	4.08	0.37	0			0
Column 3	14.5 long		1.12	0.035	9.90	0.35	14.5	0.61	3.47	4.08	0.37	0	20.0	14.0	14.0
Front wall	(0.92)(17.15)(18.0)	289.8	43.47	1.350	11.42	15.42	17.50	34.45	176.06	210.76	0.92	0.10	21.29	611.91	612.01
Back wall	(0.92)(17.15)(18.0)	289.8	43.47	1.350	11.42	15.42	17.50	34.45	176.06	210.76	0.92	0.10	21.29	611.91	612.01
Column strap footing	(2.5)(2.67)(41.6)	277.7	41.66	1.244	1.34	1.73	2.67	0.76	2.32	3.08	41.66	187.1			187.1
Front wall footing	(2.67)(2.0)(18.0)	96.12	14.42	0.448	1.34	0.60	2.67	0.27	0.80	0.05	2.0	0.15	21.5	207.09	207.24
Back wall footing	(2.67)(2.0)(18.0)	96.12	14.42	0.448	1.34	0.60	2.67	0.27	0.80	0.05	2.0	0.15	21.5	207.09	207.24
Floor	(0.5)(41.66)(15.5)	322.8	48.42	1.504	2.42	3.64	0.5	0.03	8.81	9.12	41.66	217.5			217.5
Totals				8.18	8.83	72.21				1144.25					2303.49

i. Polar Moment of Inertia I_o. In table 7.14 the polar moment of inertia of the total structure is obtained about point "O." The location of each element in the x and y directions is indicated in figure 7.36. The total moment of inertia is the sum, for all elements, of the moments of inertia of each element about its own axis and the moment of inertia of each element about the axis through point "O." The inertial effect of the soil between the front and rear footings is conservatively neglected. From table 7.14

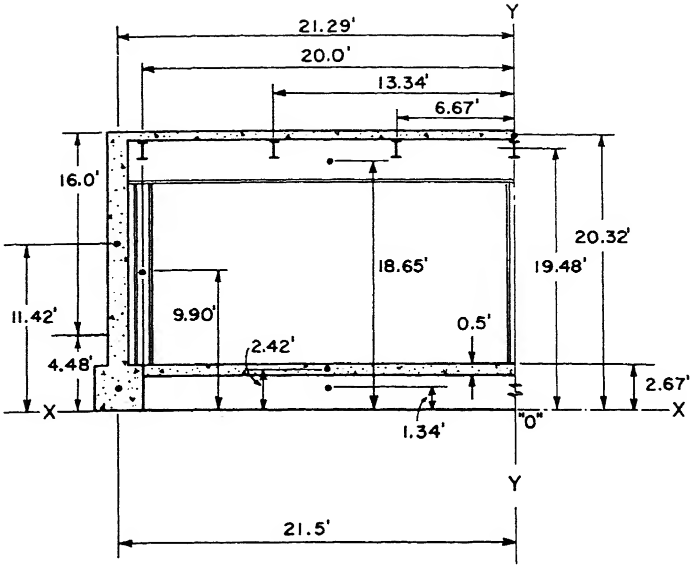


Figure 7.36. Half cross section of building

$$\bar{y} = \frac{\Sigma m\bar{y}}{\Sigma m} = \frac{72.21}{8.18} = 8.83 \text{ ft}$$

$$I_o = I_{xx} + I_{yy} = 1144 + 2304 = 3448 \text{ kip-sec}^2/\text{ft}$$

j. Ground Foundation Interaction. The rotation of the structure under the blast loads develops a resisting moment of the vertical soil pressures against the footings. The magnitude of this moment reaction which tends to resist the overturning of the structure is given by $M_\theta = B\theta$ where

$$B = \frac{\pi E}{4(1 - \nu^2)} (L)^2 b \quad \text{(eq 4.59)}$$

To apply this formula to a specific structure the ratio I_{net}/I_{gross} must be determined, where I_{net} is the moment of inertia of the actual footing area and I_{gross} is the moment of inertia of a solid rectangular area extending over the entire extent of the footings, both quantities being evaluated about the longitudinal axis of rotation (par. 4-15d). The net overturning resistance of the soil is given by the product of equation (4.59) and the ratio I_{net}/I_{gross}

$$\frac{I_{net}}{I_{gross}} = \frac{(1/12)bd^3 - (1/12)b_1d_1{}^3}{(1/12)bd^3} = 1 - \frac{b_1d_1{}^3}{bd^3}$$

where, from figure 7.35,

$b = 18.0$ ft

$d = 45.0$ ft

$b_1 = 18.0 - 2.5 = 15.5$ ft

$d_1 = 45.0 - 2(2.0) = 41.0$ ft

$L = d/2 = 45/2 = 22.5$ ft

$$\frac{I_{net}}{I_{gross}} = 1 - \frac{15.5(41.0)^3}{18.0(45.0)^3} = 0.348$$

For an 18-ft typical bay

$$B_{gross} = \frac{\pi(40)(12)^2}{4(1 - 0.333^2)} (22.5)^2\ 18 = 46.1(10)^6 \text{ kip-ft/radia}$$

$$B_{net} = (0.348)B_{gross} = (0.348)46.1(10)^6 = 16.1(10)^6 \text{ kip-ft}$$

k. Dynamic Analysis. The dynamic rigid body sliding and o analysis is performed in table 7.15. A simultaneous numerical in is performed to obtain the time history of the rigid body rotatio the rigid body translation x_o. The acceleration impulse extrapol method (eq 5.49) is used for the dynamic analysis in the form

$$(\theta)_{n+1} = 2(\theta)_n - (\theta)_{n-1} + (\alpha_o)_n(\Delta t)^2$$

$$(x_o)_{n+1} = 2(x_o)_n - (x_o)_{n-1} + (\ddot{x}_o)_n(\Delta t)^2$$

where

$$(\ddot{x}_o)_n = \frac{(F_o)_n}{m} - (\alpha_o)_n\bar{y} \text{ (eq 6.95)}$$

While the structure is sliding

$$(\alpha_o)_n = \frac{(M_o)_n - (F_o)_n\bar{y}}{I_o - m\bar{y}^2} \text{ (eq 6.94)}$$

where

$(M_o)_n$ = moment of all external forces about axis of rotat at time n .

$(F_o)_n$ = summation of all external horizontal forces appli structure at time n

Table 7.15. Simultaneous Sliding and Overturning Analysis

(1) t (sec)	(2) M'_o (kip-ft)	(3) V_F (kips)	(4) V_B (kips)	(5) M_V net (kip-ft)	(6) F_P (kips)	(7) $F_P y_{P1}$ (kip-ft)	(8) F_a (kips)	(9) $F_a y_a$ (kip-ft)	(10) $B\theta$ (kip-ft)	(11) M_o (kip-ft)	(12) P_n (kips)
Ref	table 7.13	$1.95\overline{P}_{front}$	$1.95\overline{P}_{back}$	-22.12 [(3)-(4)]	0.00158 (25)	-1.49 (6)	14.93 (4)	-2.24 (8)	-16.1 (20)	(2) + (5) + (7) + (9) + (10)	fig. 7.25
0	545	50	0	-1106	0	0	0	0	0	-561	55
0.005	1029	47	0	-1140	0.24	0	0	0	+94	+83	210
0.010	1797	41	0	-907	0.68	-1.0	0	0	+219	+1108	390
0.015	2150	37	0	-818	0.93	-1.5	0	0	+175	+1506	560
0.020	2168	32	0	-708	1.31	-2.0	0	0	-35	+1424	700
0.025	2215	29	0	-641	1.31	-2.0	0	0	-491	+1082	810
0.030	2257	25	0	-555	1.31	-2.0	0	0	-1074	+627	890
0.035	2373	23	1.0	-486	1.31	-2.0	14.93	-33	-1731	+122	960
0.040	2559	22	2.0	-442	1.31	-2.0	29.86	-67	-2401	-352	990
0.045	2708	21	3.0	-398	1.31	-2.0	44.79	-100	-3030	-821	975
0.050	2845	20	4.0	-354	1.31	-2.0	59.72	-134	-3564	-1208	950
0.055	2824	20	5.0	-332	1.31	-2.0	74.65	-167	-3957	-1633	930
0.060	2529	19	6.0	-288	1.31	-2.0	89.58	-201	-4158	-2119	910
0.065											

(13) V (kips)	(14) V (kips)	(15) H_{net} (kips)	(16) F_c (kips)	(17) $F_o\overline{y}$ (kip-ft)	(18) α_o (rad/sec^2)	(19) $\alpha_o(\Delta t)^2$ (10^{-6} rad)	(20) θ (10^{-6} rad)	(21) F_o/m (ft/sec^2)	(22) $\alpha_o\overline{y}$ (ft/sec^2)	(23) $\ddot{x}_o$ (ft/sec^2)	(24) $\ddot{x}_o(\Delta t)^2$ (10^{-6} ft)	(25) x (10^{-6} ft)
161 + (3) + (4) + (12)	0.50 (13)	(2) x table 7.12	(15) - (14) - (6) - (8)	8.83 (16)	par. 7-24k			0.122 (16)	8.83 (18)	(21) - (22)		
266	133	218	+85.0	+751	-(0.467)/2	-5.84	0	+10.37	-2.06	(+12.43)/2	155.4	0
418	209	243	+34.0	+300	-0.077	-1.93	-5.84	+4.15	-0.68	+4.83	120.8	155.4
592	296	288	-8.5	-75	+0.421	+10.53	-13.61	-1.04	+3.72	-4.76	-119.0	431.6
758	379	420	+40.0	+353	+0.410	+10.25	-10.85	+4.88	+3.62	+1.26	+31.5	588.8
893	447	414	-34.0	-300	+0.614	+15.35	+2.16	-4.15	+5.42	-9.57	-239.3	777.5
1000	500	409	0	0	+0.314	+7.85	+30.52	0				726.9
1076	538	403	0	0	+0.182	+4.55	+66.73	0				
1145	573	400	0	0	+0.035	+0.88	+107.49	0				
1175	588	397	0	0	-0.102	-2.55	+149.13	0				
1160	580	397	0	0	-0.238	-5.95	+188.22	0				
1135	568	397	0	0	-0.350	-8.75	+221.36	0				
1116	558	382	0	0	-0.474	-11.85	+245.75	0				
1096	548	355	0	0	-0.615	-15.38	+258.29	0				
							+255.45	0				

While the structure is not sliding

$$(\alpha_o)_n = \frac{(M_o)_n}{I_o} \text{ (eq 6.96)}$$

In table 7.15, M_o' is the summation of external moments about point of rotation excluding horizontal footing projection moments. blast loads on front and back footing projections are obtained by pressure values from figures 7.9 and 7.10, respectively, by $9/12[144\overline{P}(18)/1000] = 1.95\overline{P}$.

The lateral passive soil pressure force F_P is considered to linearly with displacement from zero to a peak value at $x = 0.04H_B = 0.04(4.48) = 0.179$ ft.

From paragraph 7-27 the maximum available force is $F_P = 181 = 283$ kips, where passive pressure on rear face gives [18(10)(0.1)($= 181$ and friction between wall footings gives 0.5(0.1)15.5(4.48 - $= 102$. Thus at any displacement x, the passive force is $F_P = (28 = 1580x$. This is the value with no pressure on the soil at the rea building. If there is blast pressure on the soil at the rear, the force increases. The additional passive force can be determined fr surcharge equivalent of the overpressure, thus $\Delta F_P = K_{p\phi}$ (blast loa $=[10(144)\overline{P}_{back}(18)4.48]/1000 = 116\overline{P}_{back}$. In table 7.15 the lateral ceases before the blast wave begins to load the soil at the rear of building and the passive force is considered to remain at a constan

The effect of the pressure on the soil is then considered as tive soil pressure force equal to one-fourth the passive force, or

$$\frac{116\overline{P}_{back}}{4} = 29\overline{P}_{back}$$

The centroid of F_P is at $y_{pl} = 4.48/3 = 1.49$ ft above the base of t ing, and the centroid of the blast pressure induced force is at $y_a = 4.48/2 = 2.24$ ft above the base of the footing.

The vertical load is obtained by adding the dead load of the foundation, and columns to the tabulated values of P_n and the blast on front and back footing projections. The partial dead load obtai table 7.14 is:

Columns	3.4
Front wall	43.5
Back wall	43.5
Column strap	41.7
Front wall footing	14.4
Back wall footing	14.4
	160.9 kips

In table 7.15 column 18, when $x_{n+1} > x_n$, i.e. structure is sliding,

$$(\alpha_o)_n = \frac{(M_o)_n - (F_o)_n\bar{y}}{I_o - m\bar{y}^2} = \frac{\text{column 11 - column 17}}{3448 - 8.18(8.83)^2}$$

$$(\alpha_o)_n = 0.0003559 \text{ (column 11 - column 17)}$$

When $x_{n+1} < x_n$, i.e. structure is not sliding,

$$(\alpha_o)_n = \frac{(M_o)_n}{I_o} = \frac{\text{column 11}}{3448} = 0.00029 \text{ (column 11)}$$

The results of the numerical analysis in table 7.15 are:

(1) Maximum rigid-body translation $(x_o)_m = 0.000778$ ft,

(2) Maximum rigid-body rotation $(\theta)_{max} = 0.000258$ radians,

(3) Vertical upward motion at the front = 0.000258(45/2) = 0.0058 ft.

These motions are negligible; the next step is to investigate soil pressures and member sizes.

1. Design of Foundation Members. In a refined analysis it would be necessary to determine stresses at various times in order to determine the critical stress condition. However, to illustrate the procedure in this design, stresses are computed at only one time.

The column strap footing is designed for the stress conditions at t = 0.060 sec. One bay of the foundation 18 ft wide centered on a column strap footing is considered.

The soil pressure is computed from $\frac{P}{A} + \frac{\theta Bc}{I}$

where

P = total vertical load

A = total area of foundation = 2(2.0)(18) + 2.5(41.0) = 174.5 sq ft

θB = 4158 kip-ft overturning reaction (table 7.15)

c = distance to extreme point on foundation = 45.0/2 = 22.5 ft

I = moment of inertia of foundation

$$I = 136{,}000 - 89{,}200 = 46{,}800 \text{ ft}^4$$

$$\frac{P}{A} = \frac{1096}{174.5} = 6.27 \text{ ksf}$$

table 7.15

$$\frac{\theta B c}{I} = \frac{4158(22.5)}{46{,}800} = 2.00 \text{ ksf}$$

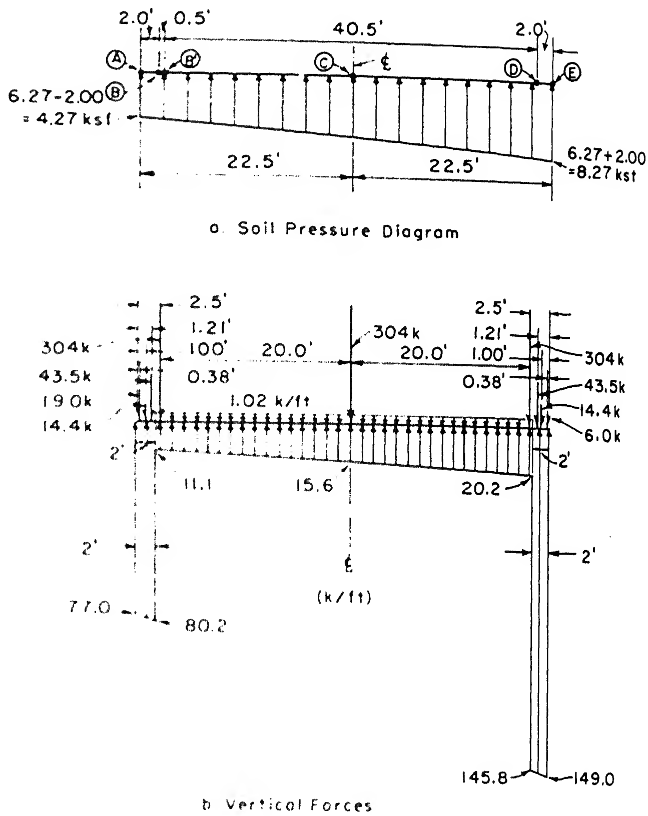


Figure 7.37. Vertical forces and soil pressure on foundation

The soil pressure multiplied by the footing width gives the per foot. In figures 7.37 and 7.38 all the forces and moments act the foundation at time t = 0.050 sec are shown. They are as foll

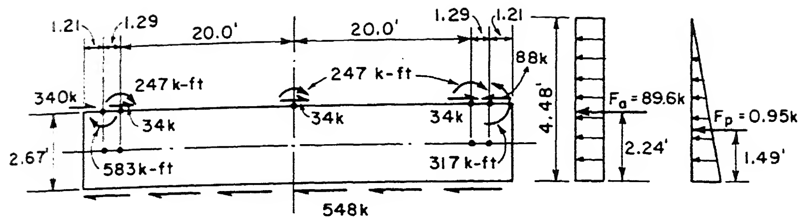
Front wall = back wall = 43.47 kips (table 7.14)

Wall footing = 14.42 kips (table 7.14)

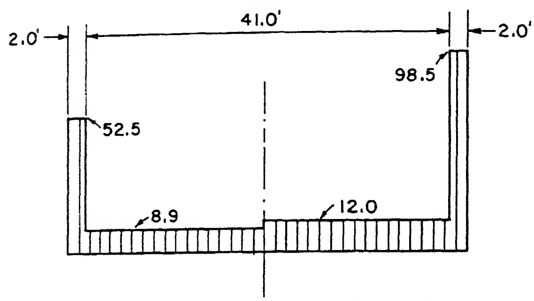
Column strap footing = 41.66/41.0 = 1.02 kips/ft (table 7.1

Front footing blast load = 19 kips (table 7.15)

Back footing blast load = 6.0 kips (table 7.15)



a. Applied Moments and Lateral Shears



b. Moment (kip-ft/ft) about Centerline of Footing Due to Soil Friction

Figure 7.38. Applied moments and shears and moment due to friction

Column load = (3.35 + 910)/3 = 304 kips (table 7.15)

Moment at base of front wall = 583 kip-ft (table 7.13)

Moment at base of back wall = 317 kip-ft (table 7.13)

Shear at base of front wall = $18V_{2n}$ = 18(18.9) = 340 kips (table 7.2)

Shear at base of back wall = $18V'_{2n}$ = 18(4.9) = 88 kips (table 7.11)

Friction at base of footing = 548 kips (table 7.15)

Column shear = 102/3 = 34 kips (fig. 7.29)

Column base moments = 741/3 = 247 kip-ft (table 7.13)

The moments at the base of the front and back walls are assumed to be transmitted to the column strap footing. This is a conservative procedure for the strap footing since the fixity at the base of the wall must be provided continuously along the wall.

The friction force is assumed to be distributed along the base in proportion to the total vertical soil pressure. For example, between points A and B

$$W = \frac{77.0 + 80.2}{2}(2.0) = 157.2 \text{ kips}$$

$$\text{Friction} = \frac{157.2}{1096}(548) = 78.8 \text{ kips}$$

In figure 7.38b the moment about the centerline of footi soil friction is equal to 2.67/2 (friction force per foot). T A and B the moment equals $78.8/2 \times 2.67/2 = 52.5$ kip-ft/ft. T sections are considered below.

<u>Shear Point C</u>

$$V = \Sigma \text{ loads down} - \Sigma \text{ loads up (fig. 7.37b)}$$

$$V = [6.0 + 14.4 + 43.5 + 304 + 20.5(1.02)] - \left[\frac{149.0 + 145.8}{2}(2.0) + \frac{15.6 + 20.2}{2}(20.5)\right]$$

$$V = (388.8 - 661.8) = -273 \text{ kips}$$

<u>Shear Point D</u>

$$V = [6.0 + 14.4 + 43.5] - \frac{149.0 + 145.8}{2}(2.0)$$

$$= 63.9 - 294.8 = -230.9 \text{ kips}$$

<u>Moment Point B'</u>

$$M = 583 + 340(1.34) + 247 + 34(1.34) + 52.5(2.0) + 8.9(0.$$

$$\frac{77.0 + 80.2}{2}(2.0)(1.5) + 11.1(0.5)0.25 - 19.0(2.12) -$$

$$43.5(1.29)$$

$$M = 1559.8 \text{ kip-ft}$$

<u>Moment Point C</u>

$$M = 583 + 340(1.34) + 247 + 34(1.34) + 247 + 34(1.34) + 5$$

$$8.9(20.5) + \frac{77.0 + 80.2}{2}(2.0)21.5 + \frac{11.1 + 15.6}{2}(20.5$$

$$19(22.12) - 14.4(21.5) - 43.5(21.29) - 304(20.0) -$$

$$1.02(20.5)\left(\frac{20.5}{2}\right)$$

$$M = 145.8 \text{ kip-ft}$$

An approximate check is made to determine if the sizes as be reinforced sufficiently. The bending moment capacity of the strap is based on equation (4.16) with f_{dy} and f'_{dc} replaced by respectively, because the soil pressure loads are considered as loads.

<u>Section at Point B'</u> b = 2.5 ft, assume d = 30 in.

$$M_p = pbd\, f_y d \left(1 - \frac{pf_y}{1.7f'_c}\right) \quad \text{(eq 4.16)}$$

For p = 0.020

$$M_p = 0.020(2.5)30(40)30 \left[1 - \frac{0.020(40)}{1.7(3.0)}\right]$$

M_p = 1518 kip-ft, 1518 ≈ 1560 ft-kips; OK, section can be reinforced

Shear is investigated at point D. The cross section is the same as at point B. If the initial blast load is from the opposite direction the maximum shear occurs at point B(V = 231 kips)

$$\text{Shear stress} = v = \frac{8V}{7bd} \quad \text{(eq 4.23)}$$

$$v = \frac{8(231)1000}{7(30)30} = 293 \text{ psi}$$

If no web reinforcement is used the

allowable $v = 0.04f'_c + 5000p$ (eq 4.24b)

= 0.04(3000) + 5000(0.020) = 220 psi, 220 < 293 psi; ∴ stirrups are needed to supply 293 - 220 = 73 psi

$$r = \frac{73}{40{,}000} = 0.00183$$

A_v = rbs = 0.00183(30)12 = 0.659 sq in./ft

<u>Section at Point C</u> b = 2.5 ft, assume d = 30 in.

$$M_p = pbd\, f_y d \left(1 - \frac{pf_y}{1.7f'_c}\right) \quad \text{(eq 4.16)}$$

For minimum steel, p = 0.006 (par. 4.10a)

$$M_p = 0.006(2.5)30(40)30 \left[1 - \frac{0.006(40)}{1.7(3.0)}\right]$$

M_p = 514.6 kip-ft

514.6 > 145.8 kip-ft; OK

Shear at this location = 273 kips

$$\text{Shear stress } v = \frac{8(273)1000}{7(30)30} = 347 \text{ psi}$$

$$\text{Allowable } v = 0.04f'_c + 5000p \text{ (eq 4.24a)}$$
$$= 0.04(3000) + 5000(0.006) = 150 \text{ psi}$$

Stirrups must be provided to supply (347 - 150 = 197 psi) su
web reinforcement.

$$r = 197/40{,}000 = 0.00493$$

$$A_v = rbs = 0.00493(30)12 = 1.77 \text{ sq in./ft}$$

Since the footing strap is of constant cross section in this other locations of the strap can be reinforced sufficiently. In th ticular design the earth passive pressure effect could have been ne since a much greater resistance to sliding was supplied by the fric effect of the soil. In general, however, the passive pressure effe should not be neglected because in certain cases the principal resi to sliding is supplied by the passive pressure.

The results of the dynamic analysis indicate the development maximum soil pressure which is much lower than the permissible valu the requirement of relatively high percentage of reinforcement in t column strap footing. Because of these, another cycle of design fo refinement might include a reduction in plan area of all foundation and an increased depth of column strap footing. In addition, the d foundation below ground surface could be reduced, if need be, becau use is made of the potential passive pressure.

NUMERICAL EXAMPLE, DESIGN OF A ONE-STORY STEEL FRAME BUILDING - ELASTIC BEHAVIOR

7-28 GENERAL. This numerical example presents an elastic design o typical bay of a windowless one-story, steel, rigid-frame building inforced concrete walls and roof. Included is the design of a typi slab, roof slab, girder, and column of the building (fig. 7.39).

One-way reinforced concrete slabs are used for both the wall roof deck. The roof slab is supported by structural steel purlins. girders and columns of the frame are structural steel sections join

welding to provide moment-resisting connections.

No foundation design is presented. Reference is made to paragraph 7-27 for an illustration of this design procedure.

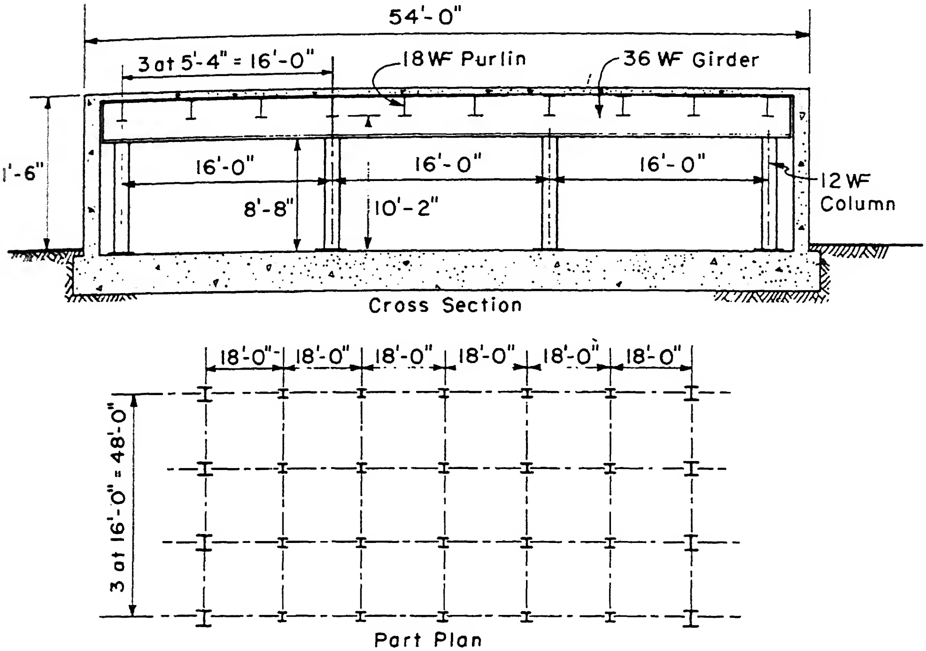


Figure 7.39. Plan and section of building

7-29 DESIGN PROCEDURE. The steps in the design procedure are as follows:

Step 1. Estimate the sizes of all structural elements which determine the over-all dimensions of the building.

Step 2. Compute and plot the time relationship for the incident overpressure, the front face overpressure, the rear face overpressure, the net lateral overpressure, and the average roof overpressure in accordance with EM 1110-345-413 procedures.

Step 3. Design the wall slab using the procedure of paragraph 6-12 and an idealized triangular load vs time curve derived from the front face overpressure vs time curve. Check the design using the actual time variation of front face overpressure in a numerical integration. From this analysis obtain the dynamic reaction on the roof slab.

Step 4. Follow the same procedure as for step 3 for the r and the purlin design. In both preliminary designs the incident pressure is idealized by a triangular load-time curve. The nume tegration check of the preliminary roof slab is performed using dent overpressure curve. The numerical integration check of the is based on the dynamic reactions from the slab giving due consi to the effect of blast wave direction on the purlin loads.

Step 5. Make a preliminary design of the column assuming to be infinitely rigid and neglecting the effect of axial load on column. As in all other preliminary designs, an idealized load-t is used. This load is obtained from the net lateral overpressure

Step 6. Design the girder using an idealized load derived purlin dynamic reactions and check by a numerical integration usi purlin dynamic reactions directly.

Step 7. Check the column design by a numerical integration ing the relative flexibility of the column and girder and the eff axial load on the column. For the load use the dynamic reactions wall slab.

7-30 <u>LOAD DETERMINATION.</u> The computation of loads is explained EM 1110-345-413 and illustrated again in paragraph 7-19 for a one building. In this example the necessary load curves are presente any explanation or computation. The design overpressure of 10 ps selected arbitrarily for this example.

The overpressure vs time curves that are presented are:

(1) Incident overpressure vs time (fig. 7.40)

(2) Front face overpressure vs time (fig. 7.41)

(3) Rear face overpressure vs time (fig. 7.42)

(4) Net lateral overpressure vs time (fig. 7.43)

(5) Average roof overpressure vs time (fig. 7.44)

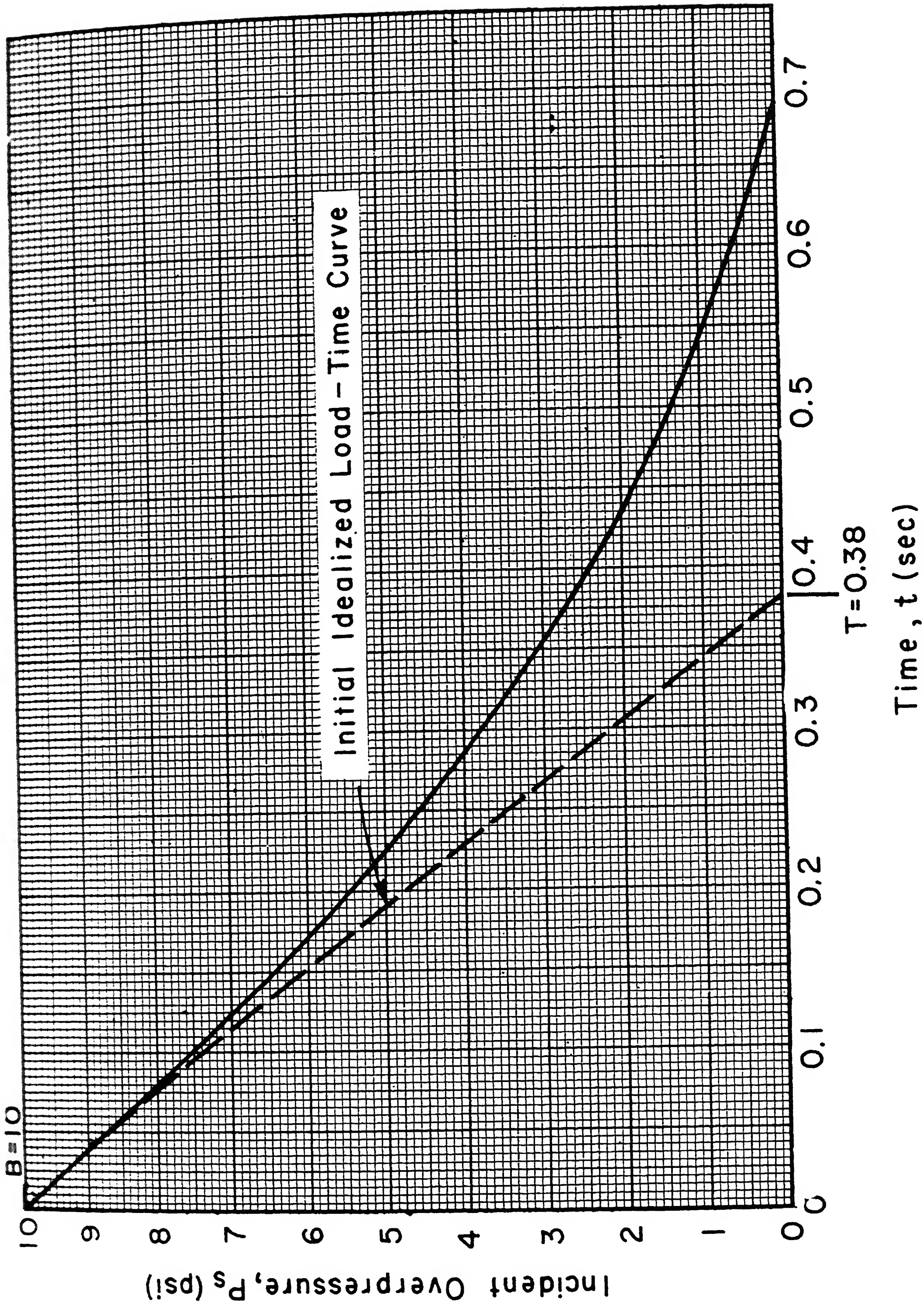


Figure 7.40. Incident overpressure vs time curve

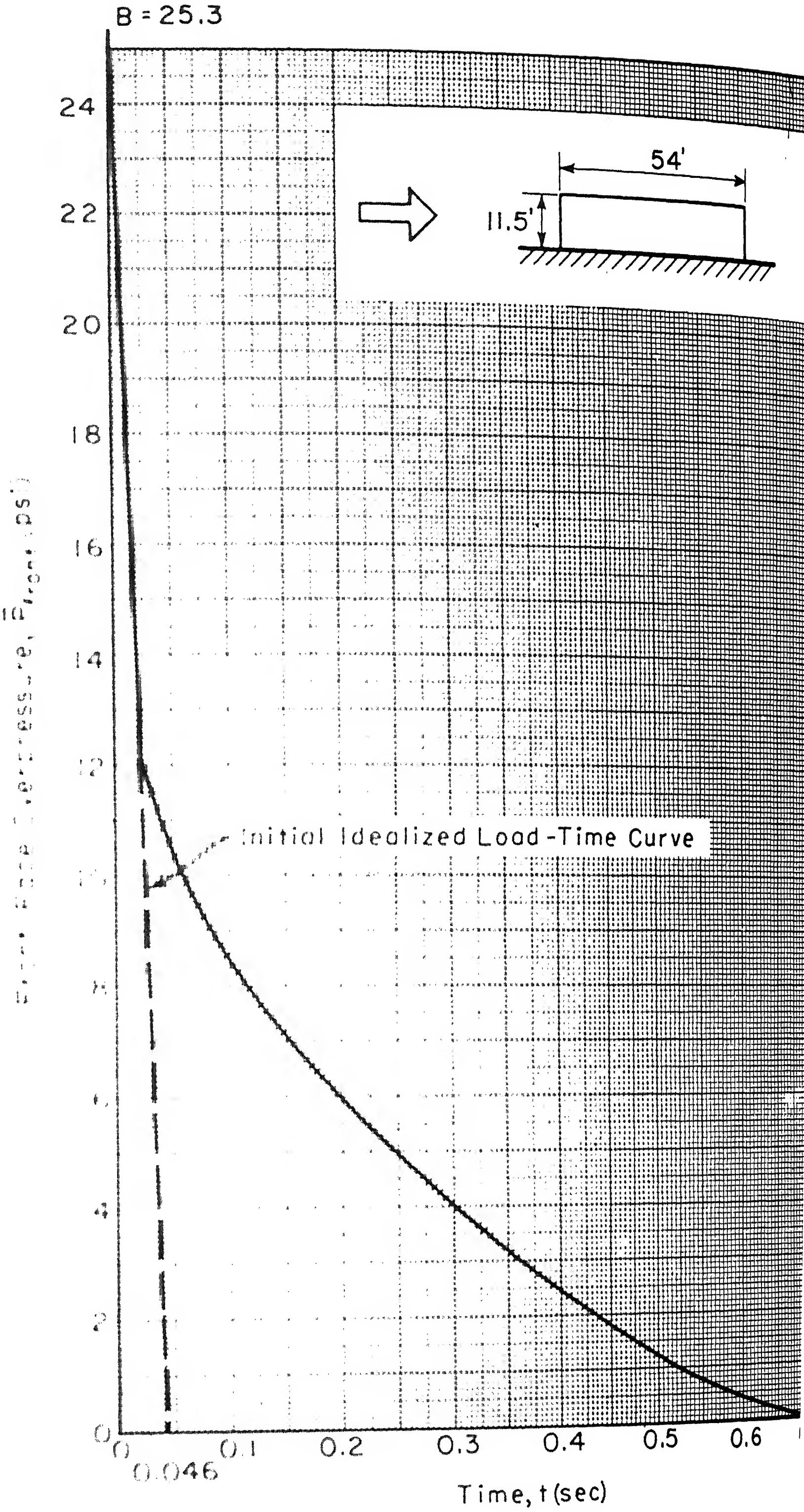


Figure 7.41. Front face overpressure vs time curve

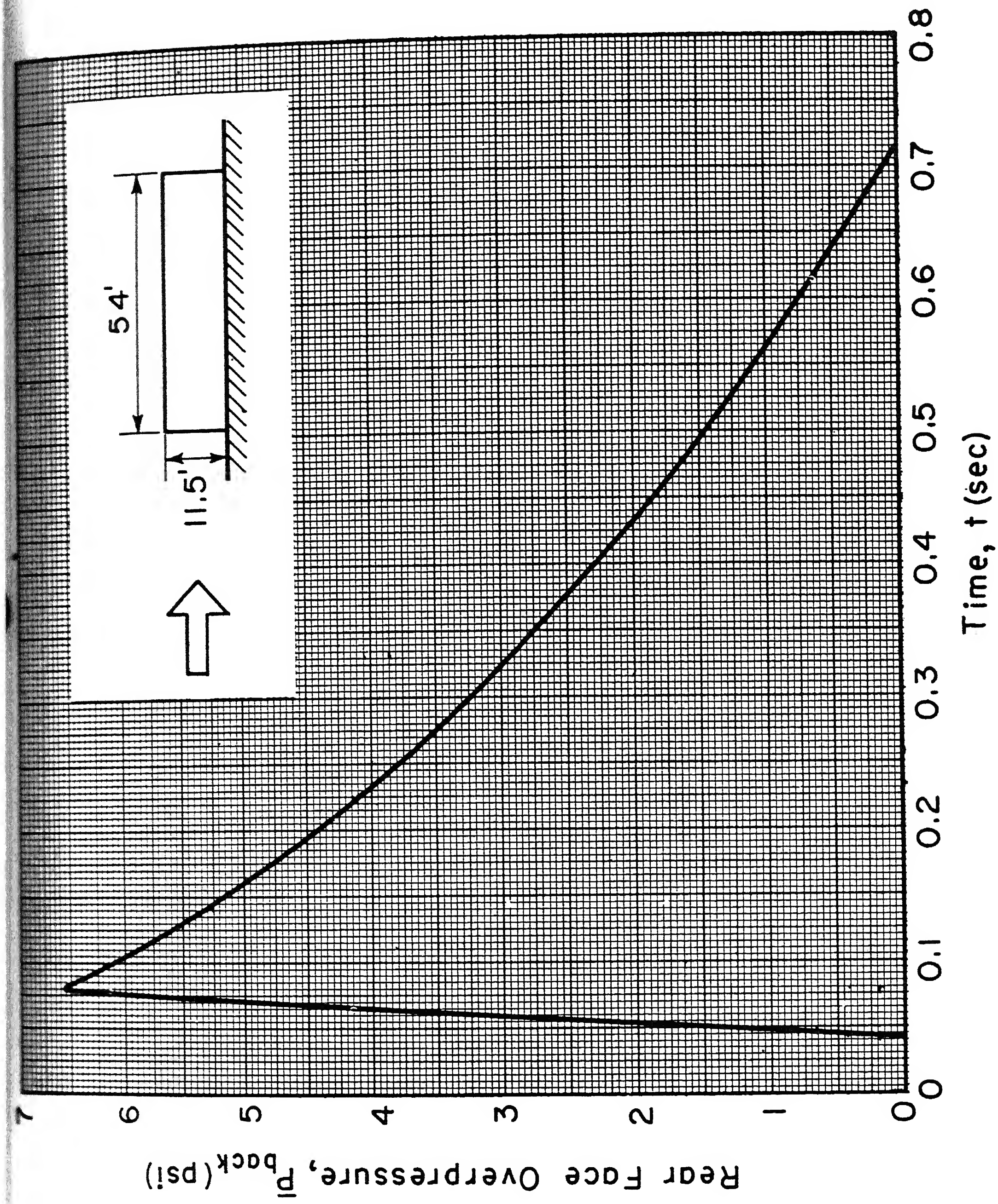
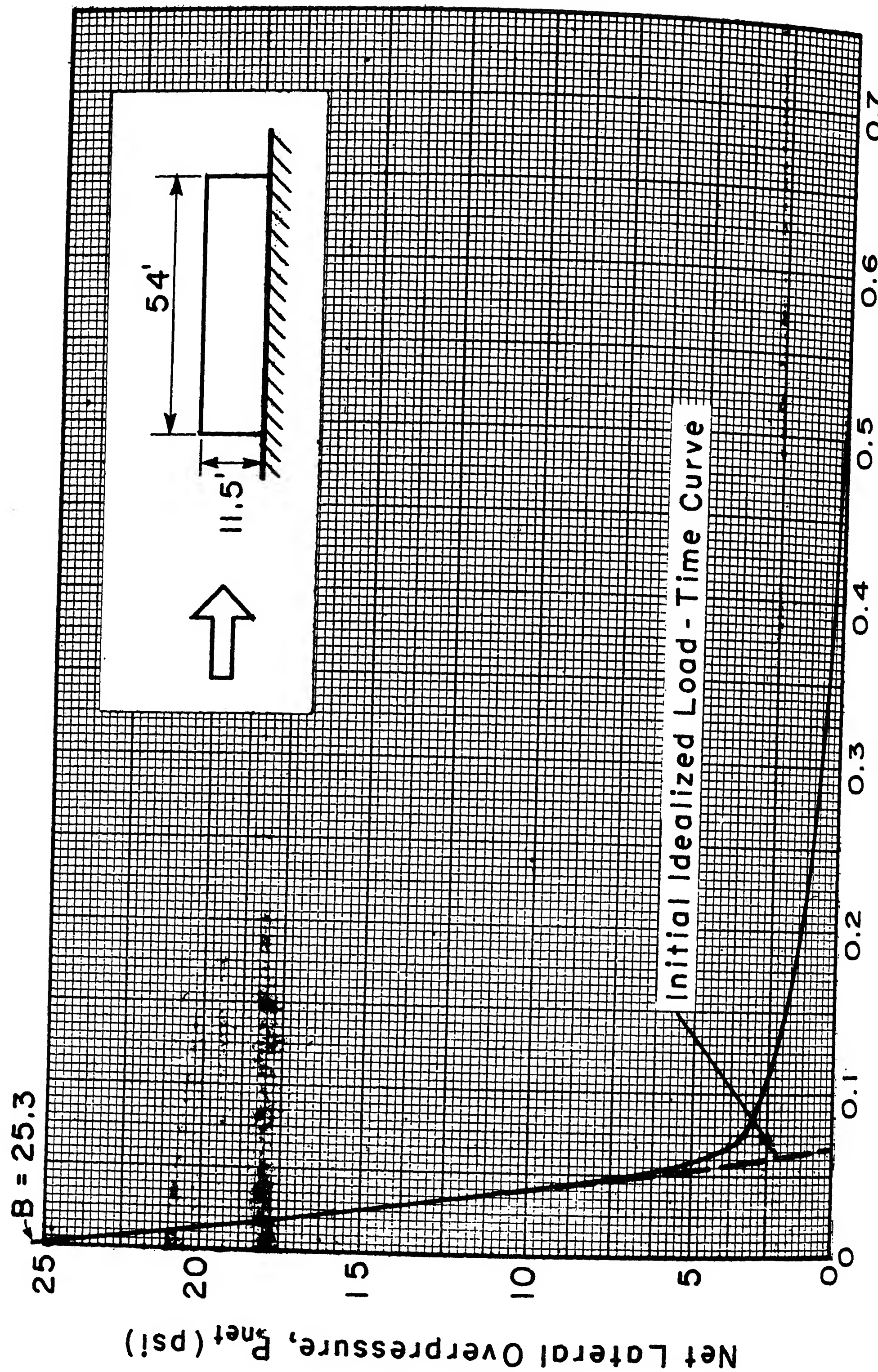


Figure 7.42. Rear face overpressure vs time curve


54'
11.5'
B = 25.3
Initial Idealized Load - Time Curve
Net Lateral Overpressure, P net (psi)
25
20
15
10
5
0
0
0.1
0.2
0.3
0.4
0.5
0.6

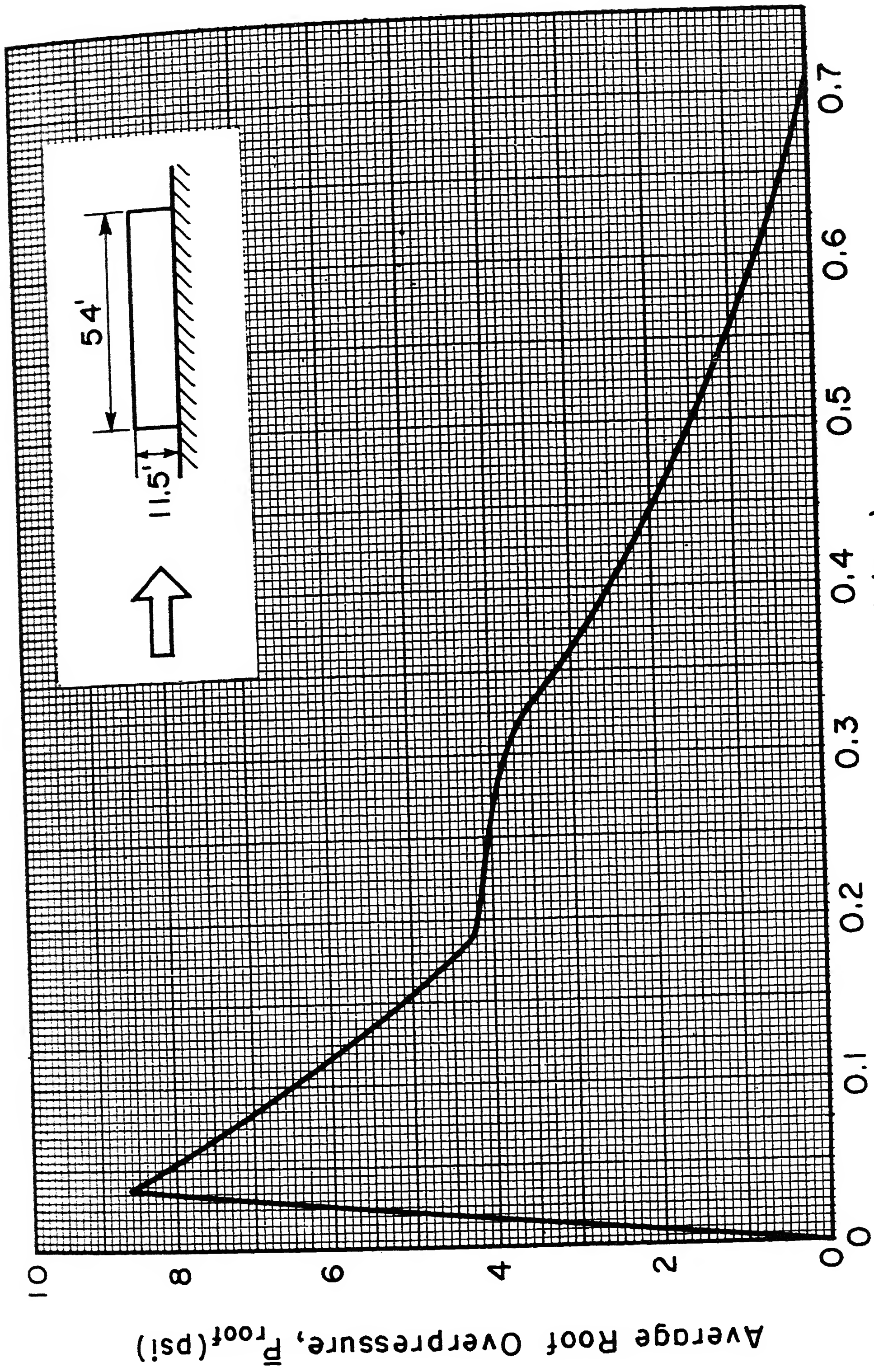
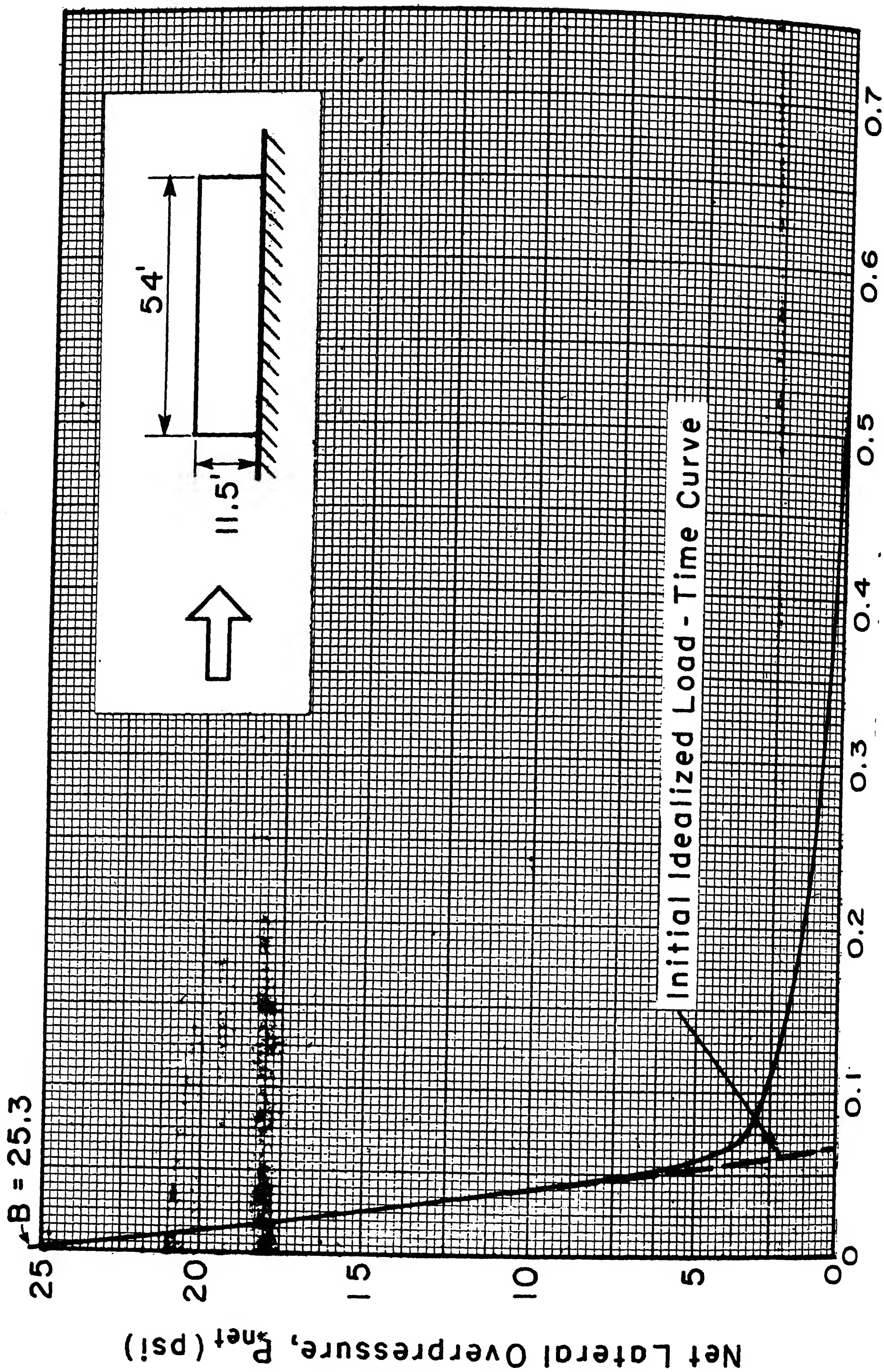


Figure 7.44. Average roof overpressure vs time curve


54'
11.5'
Initial Idealized Load - Time Curve
B = 25.3
Net Lateral Overpressure, P_{net} (psi)
25
20
15
10
5
0
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7

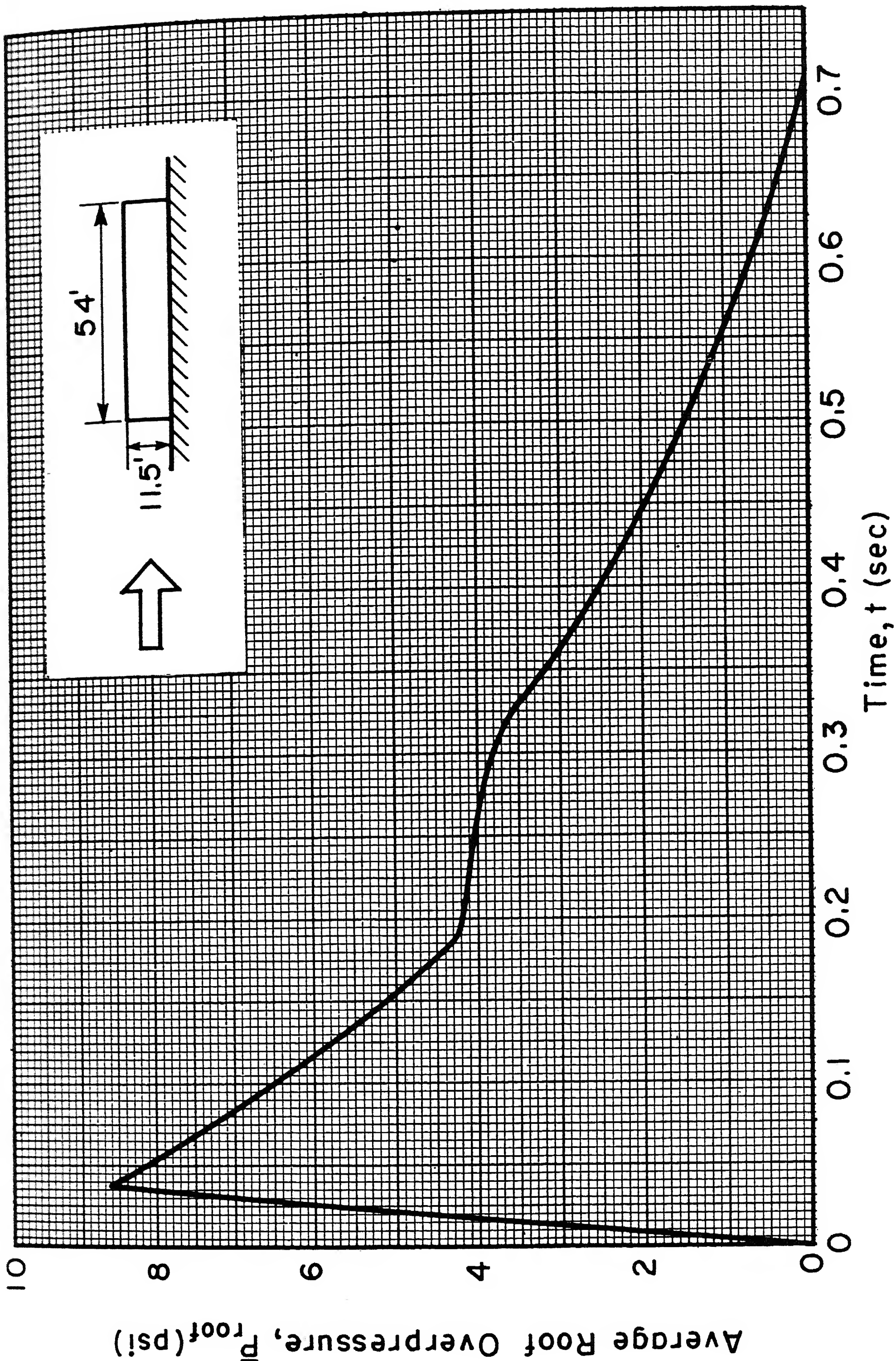
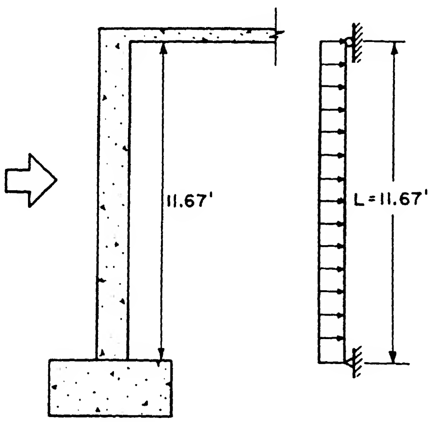


Figure 7.44. Average roof overpressure vs time curve

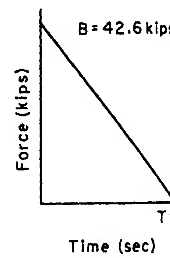
7-31 DESIGN OF WALL SLAB. The wall slab is designed as a simple foot in width, spanning vertically between the foundation and the slab. The slab is designed so that a plastic hinge will be devel midspan when the slab is subjected to the design loading. This s dition is considered to be the limit of the elastic range for pur this manual.



The preliminary elastic de cedure is outlined and then illu an example in paragraph 6-12. T load stresses are neglected for wall. The slab thickness is sel as to illustrate a procedure for slabs which are critical for she To reduce the shear stress a thi might be used.

The design span length is the clear height of the wall between foundation and roof.

a. Design Loading. The design load as idealized from the loading shown by figure 7.41 is defined by:



$$B = 25.3 \text{ psi} = \frac{25.3(11.67)(144)}{1000}$$

$$= 42.6 \text{ kips}$$

$$T = 0.046 \text{ sec}$$

b. Elastic Range Dynamic Design Factors. (Refer to table

$K_L = 0.64$, $K_M = 0.50$, $K_{LM} = 0.78$

$R_m = \dfrac{8M_P}{L}$, $k = \dfrac{384EI}{5L^3}$, $V = 0.39R$

c. First Trial - Actual Properties.

Assume D.L.F. = 1.7 (experience)

R_m = D.L.F.(B) = 1.7(42.6) = 72.5 kips (par. 6-11)

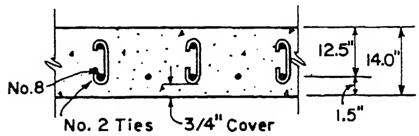
Assume p = 0.015

$$M_p = pf_{dy}bd^2\left[1 - \frac{pf_{dy}}{1.7f'_{dc}}\right] \quad \text{(eq 4.16)}$$

$$= 0.015(52)(1)d^2\left[1 - \frac{0.015(52)}{1.7(3.9)}\right] = 0.688d^2 \text{ kip-ft (d in inches)}$$

$$R_m = \frac{8M_P}{L} = \frac{(8)0.688d^2}{11.67} = 72.5, \therefore d = 12.4 \text{ in.}$$

Try $h = 14.0$ in., $d = 12.5$ in.,



$p = 0.015$, $np = 0.15$

$$M_p = 0.688d^2 = 0.688(12.5)^2 = 107.5 \text{ kip-ft}$$

$$R_m = \frac{8M_P}{L} = \frac{8(107.5)}{11.67} = 73.7 \text{ kips}$$

$$I_g = bh^3/12 = (14.0)^3 = 2740 \text{ in.}^4$$

$$I_t = bd^3\left[\frac{k^3}{3} + np(1 - k)^2\right] = 12(d)^3\left[\frac{(0.42)^3}{3} + 0.15(1 - 0.42)^2\right]$$

$$= 0.905d^3 = 0.905(12.5)^3 = 1770 \text{ in.}^4$$

$$I_a = 0.5(I_g + I_t) = 0.5(2740 + 1770) = 2255 \text{ in.}^4$$

$$k = \frac{384EI}{5L^3} = \frac{(384)3(10^3)2255}{5(11.67)^3(144)} = 2270 \text{ kips/ft}$$

$$\text{Weight} = \frac{14(150)(11.67)}{12(1000)} = 2.04 \text{ kips}$$

$$\text{Mass } m = \frac{2.04}{32.2} = 0.0634 \text{ kip-sec}^2\text{/ft}$$

d. <u>First Trial - Equivalent Properties.</u>

$k_e = K_L k = 0.64(2270) = 1450$ kips/ft (eq 6.6)

$m_e = K_M m = 0.50(0.0634) = 0.0317$ kip-sec^2/ft (eq 6.8)

$T_n = 2\pi\sqrt{m_e/k_e} = 6.28\sqrt{0.0317/1450} = 0.0293$ sec (eq 6.14)

e. <u>First Trial - Available Resistance vs Required Resistance.</u>

$C_T = T/T_n = 0.046/0.0293 = 1.57$

D.L.F. = 1.7 (fig. 5.20)

$t_m/T = 0.3$ (fig. 5.20)

$t_m = 0.3(0.046) = 0.0138$ sec

Idealized load is satisfactory replacement for actual load-t
curve up to t = 0.0138 sec (par. 5-13).

Required R_m = D.L.F.(B) = 1.7(42.6) = 72.5 kips

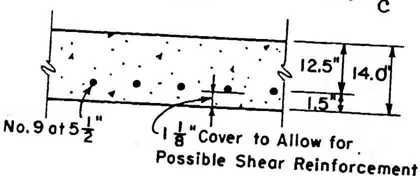
The required R_m < available R_m, therefore the selected proport
satisfactory as a preliminary design.

f. Preliminary Design for Bond Stress.

$$\text{At } t_m = 0.0138 \text{ sec, } P = \frac{(18.5)11.67(144)}{1000} = 31.0 \text{ kips (fig. 7.4}$$

$$V = 0.39R_m + 0.11P = 0.39(72.5) + 0.11(31.0) = 28.3 + 3.4 = 31$$

$$\text{Allowable } u = 0.15f'_c = 0.15(3000) = 450 \text{ psi (par. 4-09)}$$



$$\Sigma o = \frac{V}{ujd} = \frac{8(31,700)}{450(7)12.5} = 6.45 \text{ in.}$$

Try #9 at 5-1/2 in., A = 2.18 in.^{2}

Σo = 7.72 in.

$$p = A_s/bd = \frac{2.18}{12(12.5)} = 0.0145, \quad np = 10(0.0145) = 0.145$$

g. Determination of Maximum Deflection and Dynamic Reactions b
Numerical Integration.

$$I_g = bh^3/12 = (14.0)^3 = 2740 \text{ in.}^4$$

$$I_t = bd^3\left[\frac{k^3}{3} + np(1 - k)^2\right] = 12(12.5)^3\left[\frac{(0.41)^3}{3} + 0.145(1 - 0.41\right.$$

$$= 1720 \text{ in.}^4$$

$$I_a = 0.5(I_g + I_t) = 0.5(2740 + 1720) = 2230 \text{ in.}^4$$

$$\text{Weight} = \frac{14(150)11.67}{12(1000)} = 2.04 \text{ kips}$$

$$\text{Mass } m = \frac{2.04}{32.2} = 0.0634 \text{ kip-sec}^2/\text{ft}$$

$$M_P = pf_{dy}bd^2\left[1 - \frac{pf_{dy}}{1.7f'_{dc}}\right]$$

$$= (0.0145)(52)(1)(12.5)^2\left[1 - \frac{0.0145(52)}{1.7(3.9)}\right]$$

$$= 104.5 \text{ kip-ft (eq 4.16)}$$

$$R_m = \frac{8M_P}{L} - \text{weight} = \frac{8(104.5)}{11.67} - 2.04$$

$$= 71.6 - 2.04 = 69.6 \text{ kips}$$

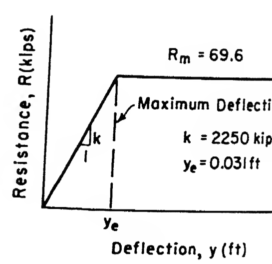


Figure 7.45. Resistance f
tion for 14-in. slab spanni
11.67 ft as a simple bean

$$k = \frac{384EI}{5L^3} = \frac{(384)3(10)^3 2230}{5(11.67)^3 144} = 2250 \text{ kips/ft}$$

$$y_e = R_m/k = \frac{69.6}{2250} = 0.031 \text{ ft}$$

Table 7.16. Determination of Maximum Deflection and Dynamic Reactions for Front Wall Slab

t (sec)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)	V_n (kips)
0	42.6	0	21.3	0.00388	0	4.9
0.003	39.5	8.7	30.8	0.00561	0.00388	7.7
0.006	36.6	30.1	6.5	0.00118	0.01337	15.7
0.009	33.7	54.1	-20.4	-0.00371	0.02404	24.8
0.012	30.8	69.6	-38.8	-0.00834	0.03100*	30.1
0.015	27.8	66.7	-38.9	-0.00834	0.02962	29.0
0.018	25.0	24.8	+0.2	+0.00004	0.0199	12.4

* $(y_n)_{max} = 0.031$ ft.

The basic equation for the numerical integration in table 7.16 is $y_{n+1} = \ddot{y}_n(\Delta t)^2 + 2y_n - y_{n-1}$ (table 5.3) where

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(\Delta t)^2}{K_{LM}(m)} = \frac{(P_n - R_n)(0.003)^2}{0.78(0.0634)} = 1.818(10^{-4})(P_n - R_n) \text{ ft}$$

The time interval $\Delta t = 0.003$ sec is approximately equal to $T_n/10$ (par. 5-08). The dynamic reaction equation is given in paragraph 7-31b. The P_n values for column 2 are obtained from figure 7.41, multiplying by 144(11.67)/1000 = 1.68 to obtain load in kips.

The maximum deflection $(y_n)_{max}$, computed in table 7.16, is 0.031 ft which is equal to the allowable y_m of 0.031 ft.

h. <u>Shear Strength and Bond Stress.</u>

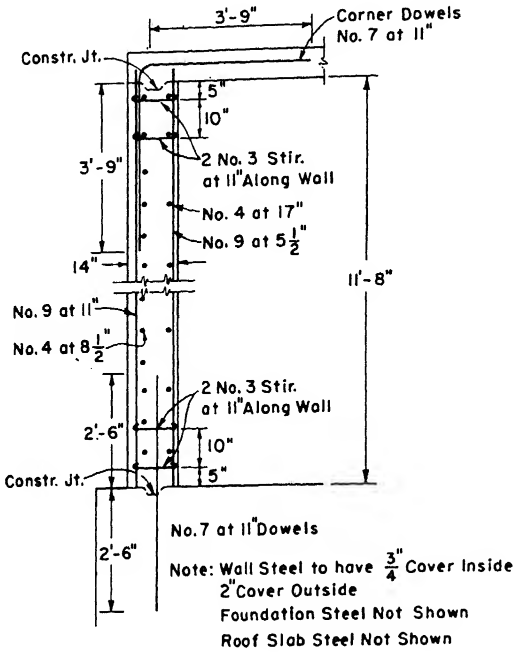
$V_{max} = 30.1$ kips (table 7.16)

For no shear reinforcement

Allowable $v_y = 0.04f'_c + 5000p$ (eq 4.24)

$v_y = 0.04(3000) + 5000(0.0145) = 120 + 72.5 = 192.5$ psi

$v = \frac{8V}{7bd} = \frac{8(30.1)}{7(12)(12.5)} = 229$ psi, therefore shear reinforcement required for 229 - 192.5 = 37 psi.



$$r = \frac{37}{40,000} = 0.001$$

Try 1 #3, $A_s = 0.11$

$$r = A_s/bs = \frac{0.11}{11s} = 0.001,$$

$$\therefore s = 10 \text{ in.}$$

$$u = \frac{8V}{7\Sigma od} = \frac{8(30,100)}{7(7.72)12.5}$$

$$= 356 \text{ psi; OK}$$

Allowable $u = 0.15f'_c$

$$= 0.15(3000) = 450 \text{ psi}$$

i. Summary.

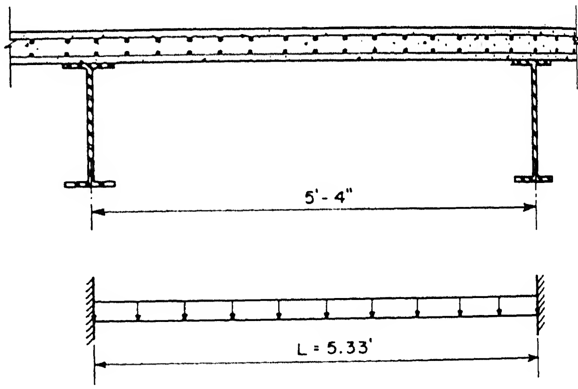
14-in. slab

$p = 0.0145$

$\Sigma o = 7.72$ in.

Shear reinforcement #3 ties at 10-in. alternate bars.

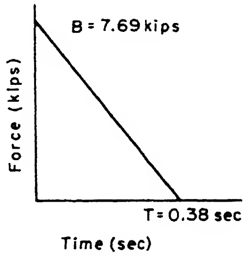
7-32 DESIGN OF ROOF SLAB. The design of the roof slab is similar to the design of the wall slab in paragraph 7-31 except for the load considerations and the inclusion of dead load stresses.



This roof slab is supported on purlins spaced at 5 ft 4 in. so as to load the girder at the third points. The design load is based on the incident overpressure curve (fig. 7.40) which results from the blast wave moving normal to the span direction of the slab. The load is uniformly distributed along the slab and varying with time. Adjacent slab elements are loaded progressively; however, the shock wave speed is such that the entire slab may be considered to be loaded at once.

The slab is continuous over the purlins; however, it is designed as a single span fixed at both ends according to the procedures of paragraph 7-11. The span length of the beam is the centerline spacing of the purlins. The design is based on a one-foot width of slab.

a. Design Loading. The design load as idealized from the computed loading shown by figure 7.40 is defined by:



$$B = 10 \text{ psi} = \frac{10(144)5.33}{1000} = 7.69 \text{ kips}$$

$$T = 0.38 \text{ sec}$$

b. Elastic Range Dynamic Design Factors. (Refer to table 6.1.)

$K_L = 0.53$, $K_M = 0.41$, $K_{LM} = 0.77$

$R_m = \dfrac{12M_P}{L}$, $k = \dfrac{384EI}{L^3}$

$V = 0.36R_n + 0.14P_n$

c. First Trial - Actual Properties.

Assume D.L.F. = 2.0 (experience)

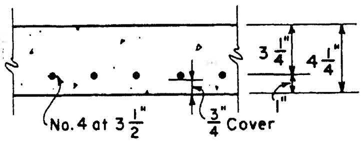
R_m = D.L.F.(B) = 2.0(7.69) = 15.38 kips (par. 6-11)

Assume p = 0.015

$$M_P = pf_{dy}bd^2\left(1 - \frac{pf_{dy}}{1.7f'_{dc}}\right) \quad \text{(eq 4.16)}$$

$$= 0.015(52)(1)d^2\left[1 - \frac{0.015(52)}{1.7(3.9)}\right] = 0.688d^2 \text{ kip-ft (d in inches)}$$

$$R_m = \frac{12M_P}{L} = \frac{(12)0.688d^2}{5.33} = 15.38, \therefore d = 3.14 \text{ in.}$$



Try h = 4-1/4 in., d = 3-1/4 in.,

p = 0.015, np = 0.15

$$M_P = 0.688d^2 = 0.688(3.25)^2 = 7.26 \text{ kip-ft}$$

$$R_m = \frac{12M_P}{L} = \frac{12(7.26)}{5.33} = 16.35 \text{ kips}$$

$$I_g = bh^3/12 = (4.25)^3 = 76.5 \text{ in.}^4$$

$$I_t = bd^3\left[\frac{k^3}{3} + np(1 - k)^2\right] = 12(d)^3\left[\frac{(0.42)^3}{3} + 0.15(1 - 0.42)^2\right]$$

$$= 0.905d^3 = 0.905(3.25)^3 = 31.0 \text{ in.}^4$$

$$I_a = 0.5(I_g + I_t) = 0.5(31.0 + 76.5) = 53.7 \text{ in.}^4$$

$$k = \frac{384EI}{L^3} = \frac{(384)3(10^3)53.7}{(5.33)^3 144} = 2840 \text{ kips/ft}$$

$$\text{Weight} = \left[\frac{4.25(150)}{12} + 6.0\right]\frac{5.33}{1000} = 0.314 \text{ kips}$$

$$\text{Mass } m = \frac{0.314}{32.2} = 0.00975 \text{ kip-sec}^2/\text{ft}$$

d. First Trial - Equivalent Properties.

$k_e = K_L k = 0.53(2840) = 1505$ kips/ft (eq 6.6)

$m_e = K_M m = 0.41(0.00975) = 0.004$ kip-sec^2/ft (eq 6.8)

$T_n = 2\pi\sqrt{m_e/k_e} = 6.28\sqrt{0.004/1505} = 0.01025$ sec (eq 6.14)

e. First Trial - Available Resistance vs Required Resistan

$C_T = T/T_n = 0.38/0.01025 = 37.0$

D.L.F. = 2.0 (fig. 5.20)

$t_m/T \approx 0$ (fig. 5.20)

$t_m \approx 0$ sec

Required R_m = D.L.F.(B) = 2.0(7.69) = 15.38 kips

The required $R_m \approx$ available R_m, therefore the selected propo... satisfactory as a preliminary design.

f. Preliminary Design for Bond Stress.

At $t_m = 0$, $P = \dfrac{(10)144(5.33)}{1000} = 7.69$ kips (fig. 7.40)

$V = 0.36R_m + 0.14P = 0.36(16.35) + 0.14(7.69) = 5.9 + 1.08 =$

Allowable $u = 0.15f'_c = 0.15(3000) = 450$ psi (par. 4-09)

$$\Sigma o = \frac{V}{ujd} = \frac{8(6980)}{450(7)(3.25)} = 5.45 \text{ in.}$$

Try #4 at 3-1/2 in., $A_s = 0.69$ in.^{2}, $\Sigma o = 5.4$ in.

$p = A_s/bd = \dfrac{0.69}{12(3.25)} = .0176$, $np = 10(.0176) = 0.176$

g. Determination of Maximum Deflection and Dynamic Reaction... Numerical Integration.

$I_g = bh^3/12 = (4.25)^3 = 76.5$ in.^{4}

$k = \sqrt{n^2p^2 + 2np} - np = 0.444$

$$I_t = bd^3\left[\frac{k^3}{3} + np(1-k)^2\right] = 12(3.25)^3\left[\frac{0.444^3}{3} + 0.176(1 - 0 \ldots\right.$$
$= 34.5$ in.^{4}

$I_a = 0.5(I_g + I_t) = 0.5(76.5 + 34.5) = 55.5$ in.^{4}

Weight = 0.314 kips

$$\text{Mass } m = \frac{0.314}{32.2} = 0.00975 \text{ kip-sec}^2/\text{ft}$$

$$M_p = pf_{dy}bd^2 \left[1 - \frac{pf_{dy}}{1.7f'_{dc}}\right]$$

$$= 0.0176(52)(1)(3.25)^2 \left[1 - \frac{0.0176(52)}{1.7(3.9)}\right]$$

$$= 835 \text{ kip-ft (eq 4.16)}$$

$$R_m = \frac{12M_p}{L} - \text{weight} = \frac{12(8.35)}{5.33} - 0.314 = 18.5 \text{ kips}$$

$$k = \frac{384EI}{L^3} = \frac{384(3)(10)^3 55.5}{5.33^3(144)} = 2930 \text{ kips/ft}$$

$$y_e = R_m/k = \frac{18.5}{2930} = 0.0063 \text{ ft}$$

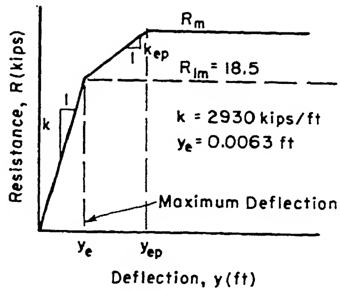


Figure 7.46. Resistance function for a 4-1/4-in. continuous slab spanning 5.33 ft

The basic equation for the numerical integration in table 7.17 is $y_{n+1} = \ddot{y}_n(\Delta t)^2 + 2y_n - y_{n-1}$ (table 5.3) where

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(\Delta t)^2}{K_{LM}(m)} = \frac{(P_n - R_n)10^{-6}}{0.77(0.00975)} = 1.33\ (10^{-4})(P_n - R_n) \text{ ft}$$

The time interval Δt = 0.001 sec is approximately equal to $T_n/10$ = 0.001025 (par. 5-08). The dynamic reaction equations are listed in

Table 7.17. Determination of Maximum Deflection and Dynamic Reactions for Roof Slab

t (sec)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)	V_n (kips)
0	7.69	0	3.84	0.00051	0	1.08
0.001	7.67	1.49	6.18	0.00082	0.00051	1.61
0.002	7.65	5.39	2.26	0.00030	0.00184	3.01
0.003	7.63	10.17	-2.54	-0.00034	0.00347	4.73
0.004	7.61	13.95	-6.34	-0.00084	0.00476	6.08
0.005	7.59	15.27	-7.68	-0.00102	0.00521*	6.56
0.006	7.57	13.60	-6.03	-0.00080	0.00464	5.96
0.007	7.54	9.58			0.00327	4.50
0.008	7.51					

* $(y_n)_{max}$ = 0.0052 ft.

paragraph 7-32b. The P_n values for the second column are obtained from figure 7.40, multiplying by $[144(5.33)(1)]/1000 = 0.769$ to obtain load in kips.

The maximum deflection $(y_n)_{max}$, computed in table 7.17, is 0.0052 ft. This is less than $y_e = 0.0063$ ft. Design is satisfactory.

h. Shear Strength and Bond Stress.

$V_{max} = 6.56$ kips (table 7.17)

For no shear reinforcement

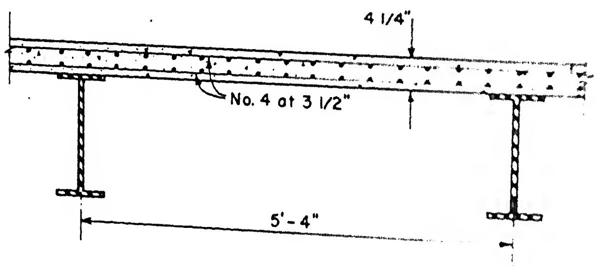
Allowable $v_y = 0.04f'_c + 5000p$ (eq 4.24)

$v_y = 0.04(3000) + 5000(0.0176) = 120 + 88 = 208$ psi

$v = \frac{8V}{7bd} = \frac{8(6560)}{7(12)(3.25)} = 192$ psi; OK, no shear reinforcement required

$u = \frac{8V}{7\Sigma od} = \frac{8(6560)}{7(5.4)3.25} = 428$ psi

Allowable $u = 0.15f'_c = 0.15(3000) = 450$ psi; OK for bond



i. Summary.

4-1/4-in. slab

$p = 0.0176$

$\Sigma o = 5.4$ in.

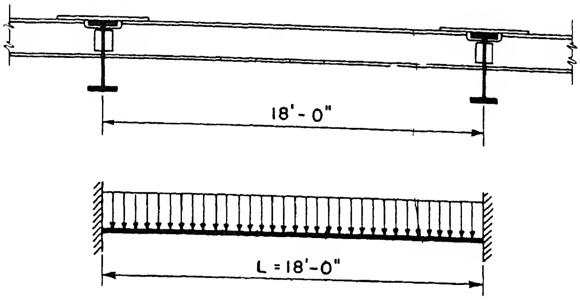
No shear reinforcement

7-33 DESIGN OF ROOF PURLINS. The purlins are framed flush with the top of the girders and are provided with moment-resisting connections. The top flanges of the purlins are anchored to the concrete roof slab to provide lateral support to the top compression flange.

Although composite behavior of the slab and purlin can be expected to develop to a limited extent, preliminary computations showed that design for independent behavior of slab and purlin is more desirable for this arrangement of slab and purlin (pars. 4-12 and 6-23). Accordingly, this purlin design neglects composite behavior.

The purlins are designed to pass into the elasto-plastic range, but not into the plastic range. This means that the deflection is limited to that necessary to just develop the plastic hinges at midspan. This point

indicated in figure 7.50 (page
). Since the design proce-
es of this manual provide only
single-span elements, this
lin is designed as a fixed-end
m spanning 18 ft between
der centerlines.



a. Loading. For the blast wave moving normal to the long axis of building and thus normal to the axis of the purlin, the loading may be sidered to be uniformly distributed along the length of the purlin. For s condition the pressure vs time variation at each point on the roof is unction of its position (par. 3-09). In addition the load on a purlin a function of the length of the slab spans between purlins because the d on the purlin increases as the blast wave traverses the two adjoining bs. In the preliminary design of the purlins the design load is ob-ned from the incident overpressure curve (fig. 7.40) without modifica-n of any sort. The rise time of the load on the slab, the slab dynamic ctions, and local variation in overpressure on the slab are all neg-ted in this preliminary step.

For the blast wave moving parallel to the long axis of the building thus parallel to the axis of the purlin, the load varies along the span a result of the time required for the blast wave to traverse the purlin n. At any point along the purlin the time variation of the load is the e and defined by the incident overpressure vs time curve (fig. 7.40).

In the calculations that follow the load vs time curves for the pur- are obtained first for the blast wave moving parallel to the long axis the building and then for the blast wave moving parallel to the short s of the building.

The purlin obtained by the preliminary design procedure is then ana-zed for both loads in tables 7.19 and 7.20 (pages 124 and 126).

ast wave moving parallel to the short axis of the building:

The variation of the local overpressure at the centerlines of pur-s (A) and (B) during the critical time period for the purlins and rders of this building is only slightly different from the incident

overpressure curve (fig. 7.40). The detailed computations to show this are not presented but are similar to the computations in paragraph 7-23a. Reference to the example in paragraph 7-23 and comparison of the critical time values of that example with the results of the following numerical analysis (table 7.18) show that the incident overpressure is a satisfactory basis for determining the purlin loading.

Table 7.18. Determination of Dynamic Reactions for Roof Slab, Local Roof Overpressure at Centerline of Purlin (A) (Including Rise Time Correction)

t (sec)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)	V_n (kips)
0	0	0	0.31	0.000041	0	0
0.001	2.01	0.12	1.89	0.000251	0.000041	0.32
0.002	4.03	0.98	3.05	0.000406	0.000333	0.91
0.003	6.04	3.02	3.02	0.000402	0.001031	1.94
0.004	7.64	6.24	1.40	0.000186	0.002131	3.32
0.005	7.62	10.01	-2.39	-0.000318	0.003417	4.67
0.006	7.60	12.85	-5.25	-0.000698	0.004385	5.69
0.007	7.57	13.64	-6.07	-0.000807	0.004655	5.97
0.008	7.55	12.07	-4.52	-0.000601	0.004118	5.41
0.009	7.53	8.73	-1.20	-0.000150	0.002980	4.19
0.010	7.51	4.93			0.001692	3.76
0.020	7.31					3.65
0.030	7.11					3.55
0.040	6.91					3.45

The roof slab is analyzed in table 7.18 for the modified incident overpressure curve presented in figure 7.47. The analysis in table 7.18 is based on the following data developed in paragraph 7.32.

Elastic range: $\ddot{y}_n(\Delta t)^2 = 1.33(10^{-4})(P_n - R_n)$ ft

$$k = 2930 \text{ kips/ft}$$

$$y_e = 0.0063 \text{ ft}$$

$$R_m = 18.5 \text{ kips}$$

$$V_n = 0.36R_n + 0.14P_n$$

This computation is performed to obtain the slab dynamic reactions on the purlins. The dynamic reactions of the slab for the overpressure variation at purlins (A) and (B) are the same for the time range considered, because

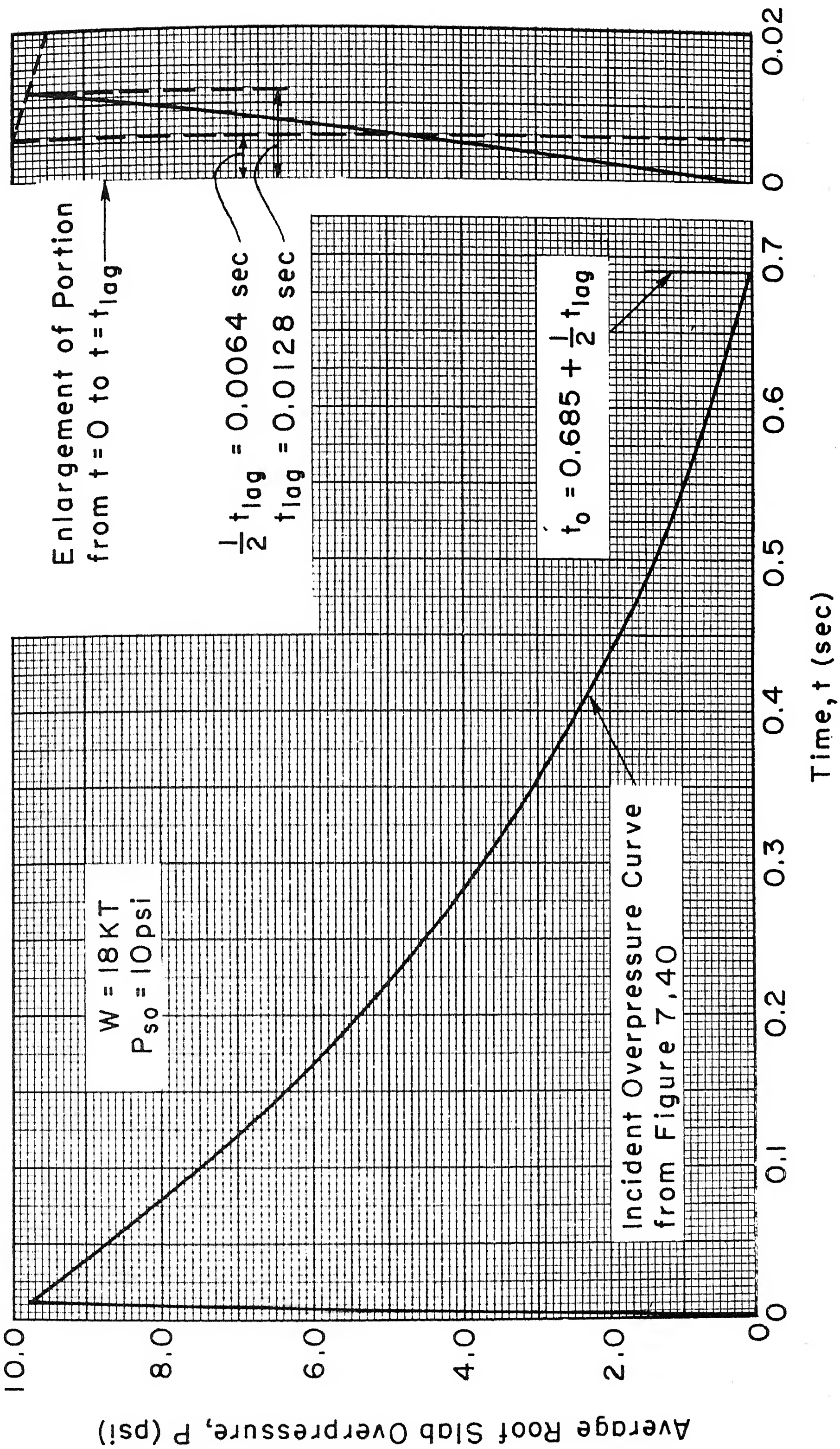


Figure 7.47. Incident overpressure curve modified for time of rise

the maximum response of the slab occurs before there is any appreciable difference in the overpressure at points (A) and (B).

To obtain the design load for purlins (A) and (B) the dynamic reaction data from table 7.18 are plotted in figure 7.48. The total purlin load is equal to the sum of the reactions of the slabs forward and aft of the purlin. In figure 7.48 it may be seen that the same dynamic reactions are plotted with a time lag

$$t_{lag} = \frac{5.33}{1403} = 0.0038 \text{ sec}$$

The loads from figure 7.48 are used in table 7.19 to check the preliminary purlin design.

Table 7.19. Determination of Maximum Deflection and Dynamic Reactions for Purlins, Blast Wave Perpendicular to Purlin Axis

t (sec)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)	V_n (kips)
0	0	0	5.6	0.00033	0	0
0.003	36.0	2.3	33.7	0.00198	0.00033	5.9
0.006	120.6	18.7	101.9	0.00600	0.00264	23.6
0.009	162.0	77.7	84.[illegible]	0.00496	0.01095	50.7
0.012	162.0	155.6	6.4	0.00037	0.02422	78.5
0.015	133.2	175.0	-41.8	-0.00243	0.03786	82.9
0.018	132.3	190.9	-58.[illegible]	-0.00341	0.04907	89.0
0.021	131.0	202.0	-71.[illegible]	-0.00413	0.05687	93.2
0.024	129.6	205.0	-75.4	[illegible]	0.06054*	93.4
0.027	128.3	193.[illegible]	[illegible]	[illegible]	0.05896	87.7
0.030	126.9	155.2	[illegible]	[illegible]	0.05352	73.6
0.033	125.6	104.7			0.04641	62.8
0.036	124.2					62.1
0.039	122.9					61.4

$*(y_n)_{max}$ = 0.0[illegible] ft.

<u>Blast wave moving parallel to the long axis of the building:</u>

The design load on the purlin is determined in figure 7.49 using the slab dynamic reactions obtained in table [illegible] for incident overpressure. The variation of the average load on the purlin with time is found by introducing the time lag

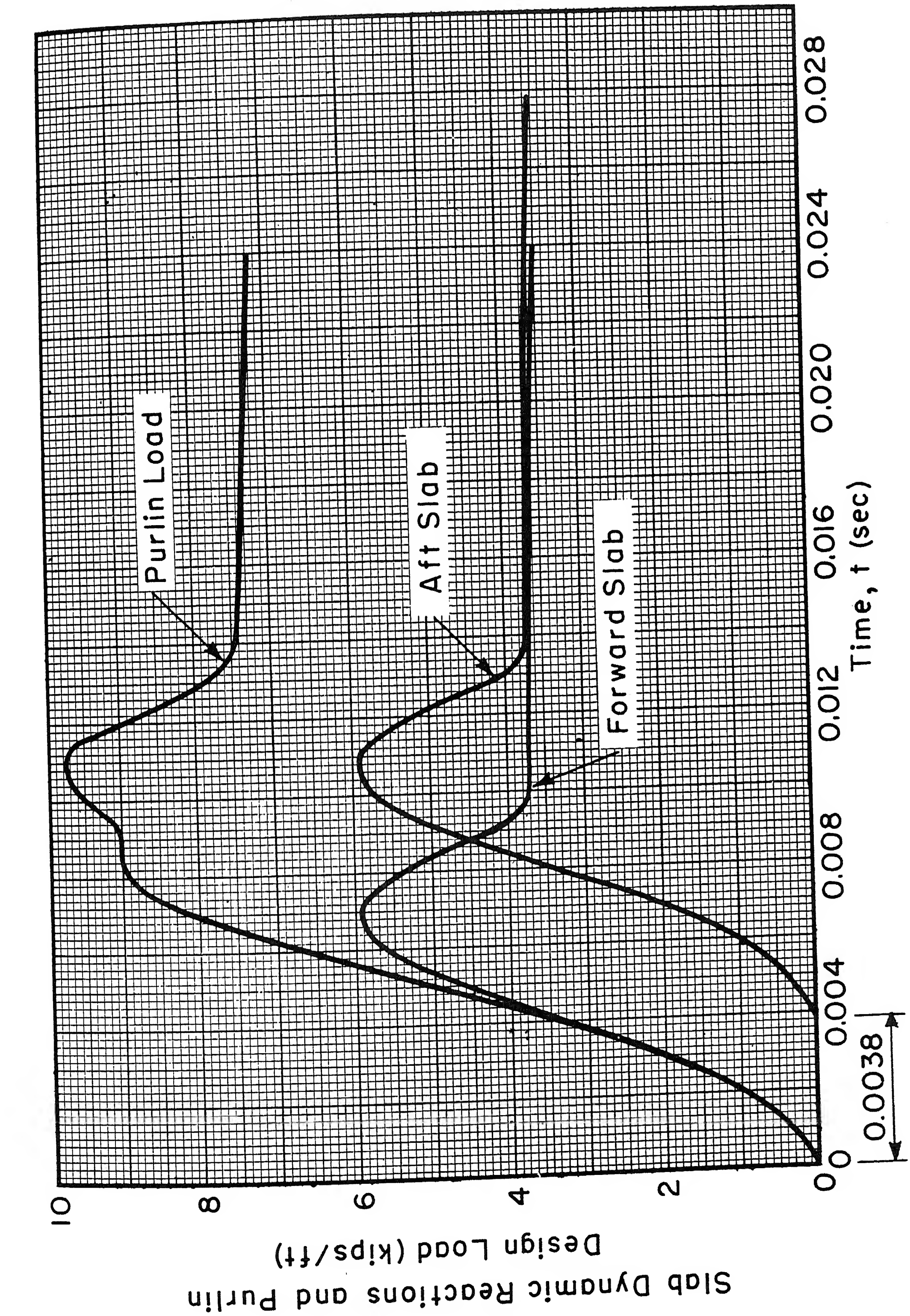


Figure 7.48. Purlin design load for local roof overpressure, blast wave moving perpendicular to purlin axis

[illegible] with time is the same at [illegible] figure 7.49 is used in [illegible] design.

[illegible] Dynamic Reactions for Purlins, [illegible] Parallel to Purlin Axis

y_n (ft)	V_n (kips)
0	0
0.00018	3.2
0.00145	13.8
.00635	31.4
.01195	57.9
.[illegible]57	78.8
.[illegible]28	83.2
.[illegible]32	87.1
.[illegible]135	90.4
.[illegible]*	91.8
.[illegible]2	86.6

[illegible] computed loading

[illegible] kips

[illegible] to table 6.

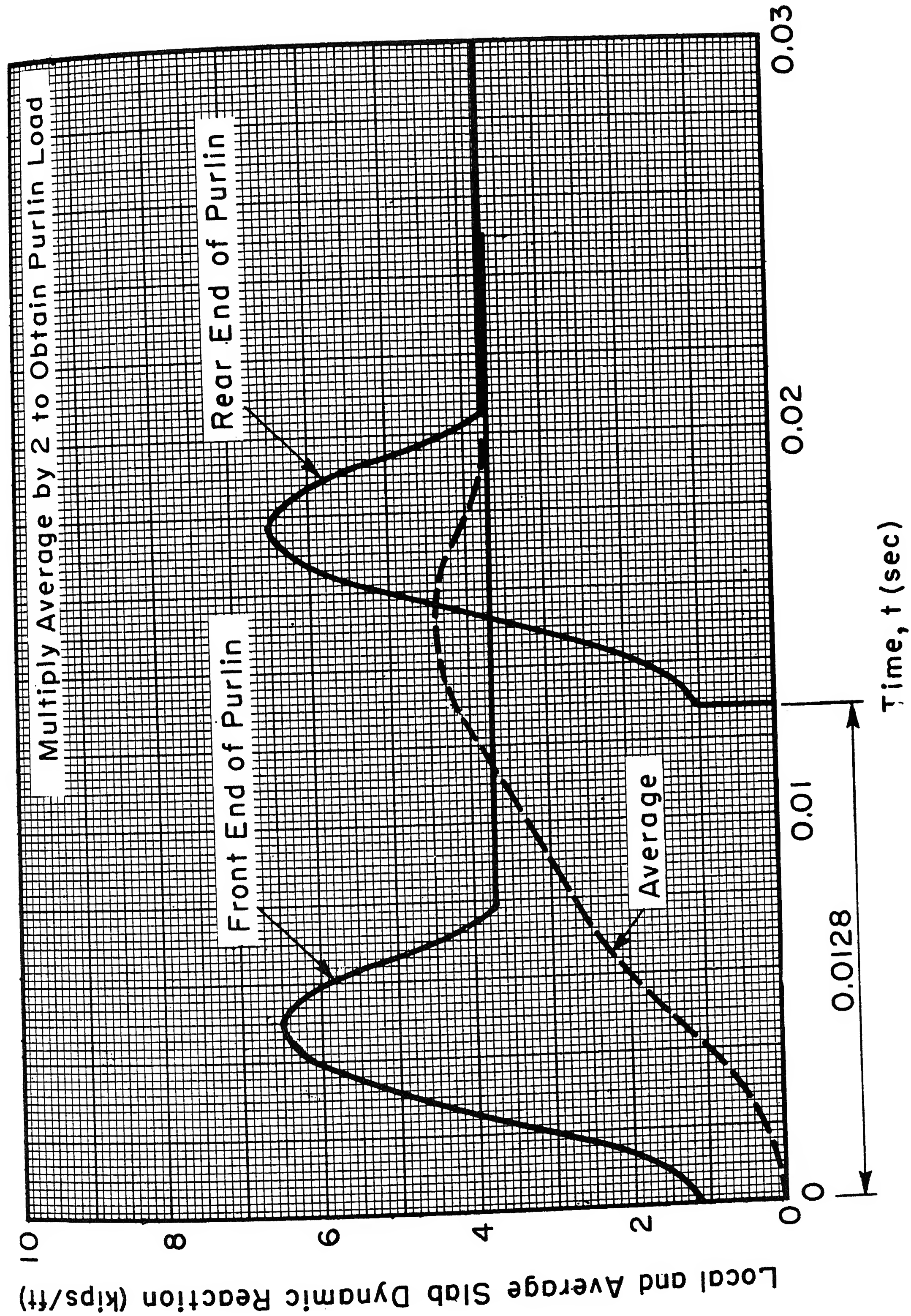


Figure 7.49. Purlin design load for incident overpressure, blast wave moving parallel to purlin axis

Elasto-plastic range:

$K_L = 0.64, \qquad K_M = 0.50, \qquad K_{LM} = 0.78$

$R_m = \frac{8}{L}(M_{Ps} + M_{Pm}), \qquad k_{ep} = \frac{384EI}{5L^3}$

$V = 0.39R + 0.11P$

Average values:

$K_L = (0.53 + 0.64)0.5 = 0.58$

$K_M = (0.41 + 0.50)0.5 = 0.45$

$K_{LM} = (0.77 + 0.78)0.5 = 0.77$

$$R_m = \frac{22M_P}{L}$$

$$k_E = \frac{264EI}{L^3}$$

c. <u>First Trial - Actual Properties.</u>

Let $M_{Ps} = M_{Pm} = M_P$

Assume D.L.F. = 2.0 and neglect dead load stresses (experience)

R_m = D.L.F.(B) = 2.0(13[illegible]) [illegible] kips (par. [illegible])

$M_P = 1.05Sf_{dy} = \frac{1.05(41.[illegible])S}{[illegible]}$ [illegible] kip-ft (S [illegible] in in.^{3}) (eq 4.2a)

$R_m = \frac{22M_P}{L} = \frac{(22)3.[illegible]S}{18}$ [illegible], ∴ S [illegible] in.^{3}

Try 16 WF 40, b/t$_f$ [illegible]; OK, S = 64.4 in.^{3}

I = 515.5 in.^{4}, Z [illegible] in.^{3}, [illegible] in.^{3}

$M_P = 0.5f_{dy}(S + Z)$ [illegible] kip-ft

$R_m = \frac{22M_P}{L} = \frac{22(237)}{18}$ [illegible]

$k = \frac{264EI}{L^3} = \frac{(264)30([illegible])}{[illegible]}$ [illegible] kips/ft

Weight = $\left\{\left[\frac{4.25([illegible])}{12} + [illegible]\right] [illegible]\right\} \frac{[illegible]}{[illegible]}$ = [illegible].4 kips

Mass m = $\frac{[illegible].4}{32.2}$ = [illegible] kip-sec^2/ft

d. <u>First Trial - [illegible]</u>

$k_e = K_Lk$ = [illegible] kips/ft (eq [illegible])

$m_e = K_M m = 0.45(0.1985) = 0.0894$ kip-sec^2/ft (eq 6.8)

$T_n = 2\pi\sqrt{m_e/k_e} = 6.28\sqrt{0.0894/2820} = 0.0353$ sec (eq 6.14)

e. First Trial - Available Resistance vs Required Resistance.

$C_T = T/T_n = 0.38/0.0353 = 10.8$

D.L.F. = 1.95 (fig. 5.20)

$t_m/T = 0.05$ (fig. 5.20)

$t_m = 0.05(0.38) = 0.019$ sec

Idealized load-time curve is satisfactory up to t_m (fig. 7.40)

Required R_m = D.L.F.(B) = 1.95(138) = 269 kips

The required R_m < available R_m; therefore the selected proportions are satisfactory as a preliminary design.

f. Preliminary Design for Shear Stress.

At $t_m = 0.019$ sec, $P = \frac{9.5(5.33)18(144)}{1000} = 131$ kips (fig. 7.40)

$V = 0.36R + 0.11P = 0.36(269) + 0.11(131) = 111.4$ kips

$v = \frac{V}{dt_w} = \frac{111{,}400}{(16)0.307} = 22{,}700$ psi (allowable v = 21,000 psi)(par. 4-05c)

Since V = 111.4 kips is estimated it is desirable to continue with investigation of 16 WF40.

g. Determination of Maximum Deflection and Dynamic Reactions by Numerical Integration.

$M_P = 0.5f_{dy}(S + Z) = 237$ kip-ft

Weight = 6.4 kips

Mass m = 0.1985 kip-sec^2/ft

Elastic range:

$$R_{1m} = \frac{12M_P}{L} - \text{weight} = \frac{12(237)}{18} - 6.4 = 151.6 \text{ kips}$$

$$k_1 = \frac{384EI}{L^3} = \frac{(384)30(10)^3 515.5}{18^3(144)} = 7100 \text{ kips/ft}$$

$$y_e = \frac{R_{1m}}{k_1} = \frac{151.6}{7100} = 0.0214 \text{ ft}$$

$$k_e = K_L k_1 = 0.53(7100) = 3760 \text{ kips/ft}$$

$$m_e = K_M m = 0.41(0.1985) = 0.0815 \text{ kip-sec}^2/\text{ft}$$

$$T_n = 2\pi\sqrt{m_e/k_e} = 6.28\sqrt{0.0815/3760} = 0.0292 \text{ sec}$$

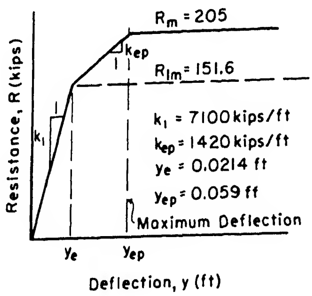


Figure 7.50. Resistance function for 16 WF 40 purlin spanning 18 ft

Elasto-plastic range:

$$R_m = \frac{16M_p}{L} - \text{weight} = \frac{16(237)}{18} - 6.4 = 205 \text{ kips}$$

$$k_{ep} = \frac{384EI}{5L^3} = \frac{1}{5}k_1 = 1420 \text{ kips/ft}$$

$$y_m = y_{ep} = y_e + \frac{R_m - R_{lm}}{k_{ep}} = 0.0214 + \frac{205 - 151.6}{1420} = 0.0214 + 0.0376 = 0.059 \text{ ft}$$

The basic equation for the numerical integration in tables 7.19 and 7.20 is $y_{n+1} = \ddot{y}_n(\Delta t)^2 + 2y_n - y_{n-1}$ (table 5.3) where

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(\Delta t)^2}{K_{LM}(m)}$$

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)9(10^{-6})}{0.77(0.1985)} = 5[illegible](10^{-6})(P_n - R_n) \text{ ft, elastic range}$$

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)9(10^{-6})}{0.78(0.1985)} = 5[illegible](10^{-6})(P_n - R_n) \text{ ft, elasto-plastic range}$$

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)9(10^{-6})}{0.66(0.1985)} = [illegible](10^{-6})(P_n - R_n) \text{ ft, plastic range}$$

The time interval $\Delta t = 0.003$ sec is approximately $T_n/10 = 0.00292$ (par. 5-08). The dynamic reaction equations are listed in paragraph 7-33b. In table 7.20 the P_n values for the second column are obtained from figure 7.49, multiplying by $2 \times 18 = 36$, to account for the two slabs loading the purlin. In table 7.19 P_n values for the second column are obtained from figure 7.48, multiplying by 18, the length of the purlin.

h. <u>Shear Stress Check.</u>

V = 93.4 kips (table 7.19)

$$v = \frac{V}{dt_w} = \frac{93{,}400}{16(0.307)} = 19{,}000 \text{ psi}$$

Allowable v = 21,000 psi; OK (par. 4-05c)

i. Check Proportions for Local Buckling. (par. 4-06d)

16 WF40, $b = 7.0$, $t_f = 0.503$

$a = 15.0$, $t_w = 0.307$

$b/t_f = 7.0/0.503 = 13.9 < 14.0$; OK

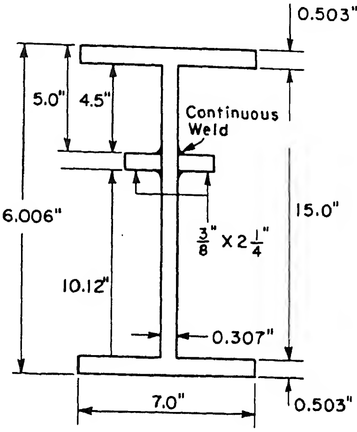
$a/t_w = 15.0/0.307 = 49.0 > 30$; NG

Longitudinal stiffeners are required

$t_s = 3/8 > 0.307$

$b_s/t_s = 6$; $b_s = 6t_s = 6(3/8) = 2.25$ in.

Use 2 plates 3/8 in. by 2-1/4 in. full length welded continuously as described in paragraph 4-06d.



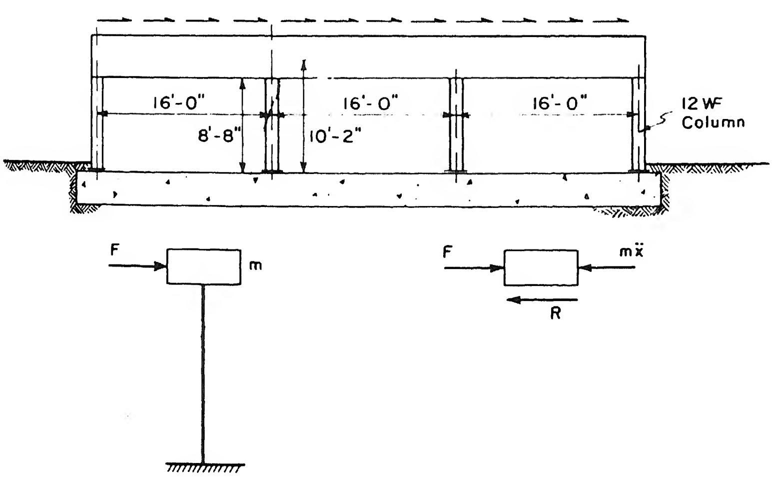
$c/t_w = (4.5)/0.307 = 14.6 < 22$; OK

$e/t_w = (10.12)/0.307 = 33 < 40$; OK

Weight of added stiffener plates = 5.7 lb/ft

7-34 PRELIMINARY DESIGN OF COLUMNS. A single-story frame subject to lateral load behaves essentially as a single-degree-of-freedom system with the column displaying the spring properties. It is therefore unnecessary to substitute an equivalent system for the original structure, and the mass and load factors that are necessary in the design of beams and slabs are not used in the design of single-story frames.

The preliminary elastic design procedure of paragraph 6-12 is used to determine the preliminary column size. The equations of paragraph 7-06 are used in the procedure of paragraph 6-12 to replace some of the factors used in determining equivalent single-degree-of-freedom systems.

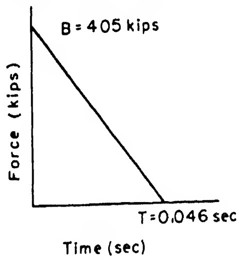


For purposes of preliminary design the frame girders are assumed to be

infinitely rigid thus simplifying the determination of the column spring constant. For spring constant computation the effective column height is 10.17 ft based on an assumed girder depth of 3 ft and clear height of 8.67 ft. The clear height is used in determining the maximum resistance of the columns (par. 7-06).

In the preliminary design of steel columns it is desirable to be conservative to allow for the factors which are neglected in the preliminary design. These factors are: (1) the effect of direct stress on the plastic hinge moment and (2) the effect of girder flexibility.

a. Design Loading. The design lateral load on the frame is obtained from the dynamic reactions at the top of the front wall slab. However, for the preliminary design procedure it is satisfactory to use the net lateral overpressure curve (fig.7.43). In this case the total concentrated lateral load on the frame is the product of ([illegible])([illegible])$\overline{P}_{net}$. The effective wall height determining the frame load is obtained from the wall clear height and roof slab thickness



$h' = \frac{[illegible]}{[illegible]} + [illegible] = [illegible]$

The design load as [illegible] from the computed loading shown by figure [illegible] is defined by

$B = 25.3$ psi $[illegible] = 405$ kips

T = 0.046 sec

b. Mass Computation.

Walls $\frac{14(11.67)2(18)150}{12(1000)} = [illegible]$ kips

Roof slab $\left[\frac{4.25(150)}{12} + [illegible]\right]\frac{[illegible]}{[illegible]} = [illegible]$ kips

Purlins $\frac{40(18)10}{1000} = 7.[illegible]$ kips

Girders (estimated) $\frac{150(49)}{1000} = 7.4$ kips

Columns (estimated) $\frac{150([illegible])4}{1000} = [illegible]$ kips

Connections = 0.1(7.2 + 7.4) = [illegible] kips ([illegible]% of girders and purlins)

Single-degree-of-freedom-system mass = total roof + girder +

$$1/3 \text{ (walls and columns)} = \frac{57.3 + 7.2 + 7.4 + 1.4 + 0.33(73.5 + 5.2)}{32.2}$$

$$= \frac{99.5}{32.2} = 3.09 \text{ kip-sec}^2/\text{ft}$$

c. <u>First Trial - Actual Properties.</u>

Assume D.L.F. = 1.2 (experience)

R_m = D.L.F.(B) = 1.2(405) = 486 kips

Required $M_D = R_m h/2n = 486(8.67)/2(4) = 527$ kip-ft (eq 7.14)

The effect of axial column load is neglected

$$M_D = f_{dy} \frac{(S + Z)}{2} \text{ (eq 4.2)}$$

$$\frac{S + Z}{2} = \frac{M_D}{f_{dy}} = \frac{527(12)}{41.6} = 152 \text{ in.}^3$$

Smallest column that satisfies buckling criteria is 12 WF 120

$S = 163.4$ in.^{3}, $Z = 186.0$ in.^{3}, $I = 1071.7$ in.^{4}, $0.5(S + Z) = 174.5$ in.^{3}

$A = 35.31$ in.^{2}

$a/t_w = 10.91/0.71 = 15.4 < 22$, OK; $b/t_f = 12.32/1.106$

$= 11.1 < 14.0$, OK (par. 4-06d)

$M_D = 0.5(S + Z)f_{dy} = (174.5)41.6/12 = 605$ kip-ft

$$R_m = \frac{2nM_D}{h} = \frac{2(4)605}{8.67} = 558 \text{ kips}$$

d. <u>First Trial - Determination of D.L.F.</u>

$$k = 12EIn/h^3 = \frac{12(30)10^3(1071.7)4}{144(10.17)^3} = 10{,}200 \text{ kips/ft (eq 7.10)}$$

$$T_n = 2\pi\sqrt{m/k} = 6.28\sqrt{3.09/10{,}200} = 0.1095 \text{ sec}$$

$$T/T_n = 0.046/0.1095 = 0.42$$

D.L.F. = 1.07, $t_m/T = 0.93$ (fig. 5.20)

$t_m = 0.93(0.046) = 0.043$ sec

The original load-time curve should be revised to obtain a closer approximation to the total impulse up to time t_m. The impulse up to $t = 0.05$ sec in figure 7.43 is:

$H = 0.82$ psi-sec (obtained by graphical integration)

$T = 2H/B = 2(0.82)/25.3 = 0.065$ sec

$T/T_n = 0.065/0.1095 = 0.593$

D.L.F. = 1.3, $t_m/T = 0.70$ (fig. 5.20)

$t_m = 0.7(0.065) = 0.0455 < 0.05$

The revised idealized load-time curve is satisfactory

Required R_m = D.L.F.(B) = 1.3(405) = 527 kips < 558 kips; OK

There is no need for further trials. The required R_m of 527 kips is greater than original estimate of 486 kips but less than R available from 12 W 120 (558 kips). Therefore the 12 W 120 is accepted for preliminary design.

e. Shear Stress Check of 12 W 120.

$$\text{Estimated } V_{max} = \frac{R_m}{4} = \frac{558}{4} = 139.5 \text{ kips}$$

$$v = \frac{V}{t_w d} = \frac{139,500}{0.710(13.12)} = 15,000 \text{ psi}$$

Allowable v = 21,000 psi; OK (par. 4-[illegible])

f. Slenderness Criterion for Beam - Column. (par. 4-08) An approximate evaluation of the 12 W 120 column slenderness criterion is made before the column size is accepted for final analysis. The criterion is:

$$\left(\frac{M_D}{M_P}\right)\left(\frac{K'Ld}{100bt}\right) + \left(\frac{P_D}{P_P}\right)\left(\frac{K''L}{15r}\right) < [illegible] \text{ (eq 4.[illegible])}$$

$M_D = \frac{527}{558}(605) = 570$ kip-ft, rough estimate based on ratio of required R_m to available R_m

$M_P = 605$ kip-ft

$P_P = f_{dy}A = 41.6(35.31) = 1469$ kips

$P_D = \frac{(8.4)144(54)18}{1000(3)} = 392$ kips, at t_m = 0.0455 sec (fig. 7.43)

$K' = 0.14$ (table 4.1)

$K'' = 0.50$ (table 4.2)

$L = 8.67$ ft (clear height)

$r = 3.13$ in. $d = 13.12$ in.

$b = 12.32$ in.

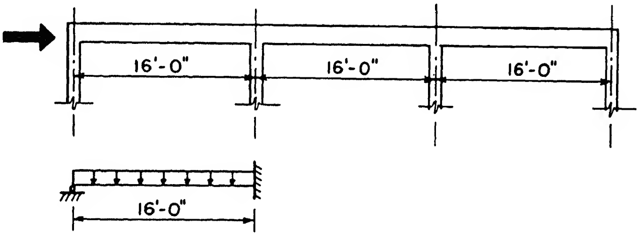
$t_f = 1.106$ in.

Substituting in equation (4.10) gives

$$\left(\frac{570}{605}\right)\left[\frac{0.14(8.67)12(13.12)}{100(12.32)(1.106)}\right] + \left(\frac{392}{1470}\right)\left[\frac{0.5(8.67)12}{15(3.13)}\right]$$

$$= 0.132 + 0.284 = 0.416 < 1; \text{ OK}$$

7-35 GIRDER DESIGN. The design procedure for the frame girder is identical to the girder design in paragraph 7-25. To provide continuity in the presentation of this building design example without excessive repetition, this girder design is limited to the computation of the resistance diagram and the numerical integration computation to check the adequacy of the girder resistance. The same preliminary computations performed in paragraph 7-25 have been performed for this girder but are omitted for simplicity of presentation. The girder is designed as a fixed-pinned beam. A 36 WF 150 girder is selected by the preliminary computations.



The design load curve (fig. 7.51) is presented. The method of determining the load curve from purlin reactions is the same as described in paragraph 7-25. The dynamic reaction data are obtained from table 7.19.

a. Determination of Maximum Deflection by Numerical Integration. Based on the preliminary computations which are omitted to reduce duplication the numerical integration analysis is performed for a 36 WF 150 girder.

$S = 503 \text{ in.}^3$, $Z = 580 \text{ in.}^3$, $I = 9012 \text{ in.}^4$

Girder $M_P = f_{dy}\,[(S + Z)/2] = 1875$ kip-ft (eq 4.2)

The static loads develop a moment at the fixed support

Static load moment = 40 kip-ft

The moment in the girder to resist lateral frame motion is

$\frac{2}{3} M_P$ of column $= \frac{2}{3}(605) = 403$ kip-ft (pars. 7-34e and 7-11)

$$R_{1m} = \frac{6M}{L} = \frac{6}{16}(1875 - 40 - 403) = 538 \text{ kips}$$

$$k = 132\frac{EI}{L^3} = \frac{132(30)10^3(9012)}{16^3(144)} = 60{,}400 \text{ kips/ft}$$

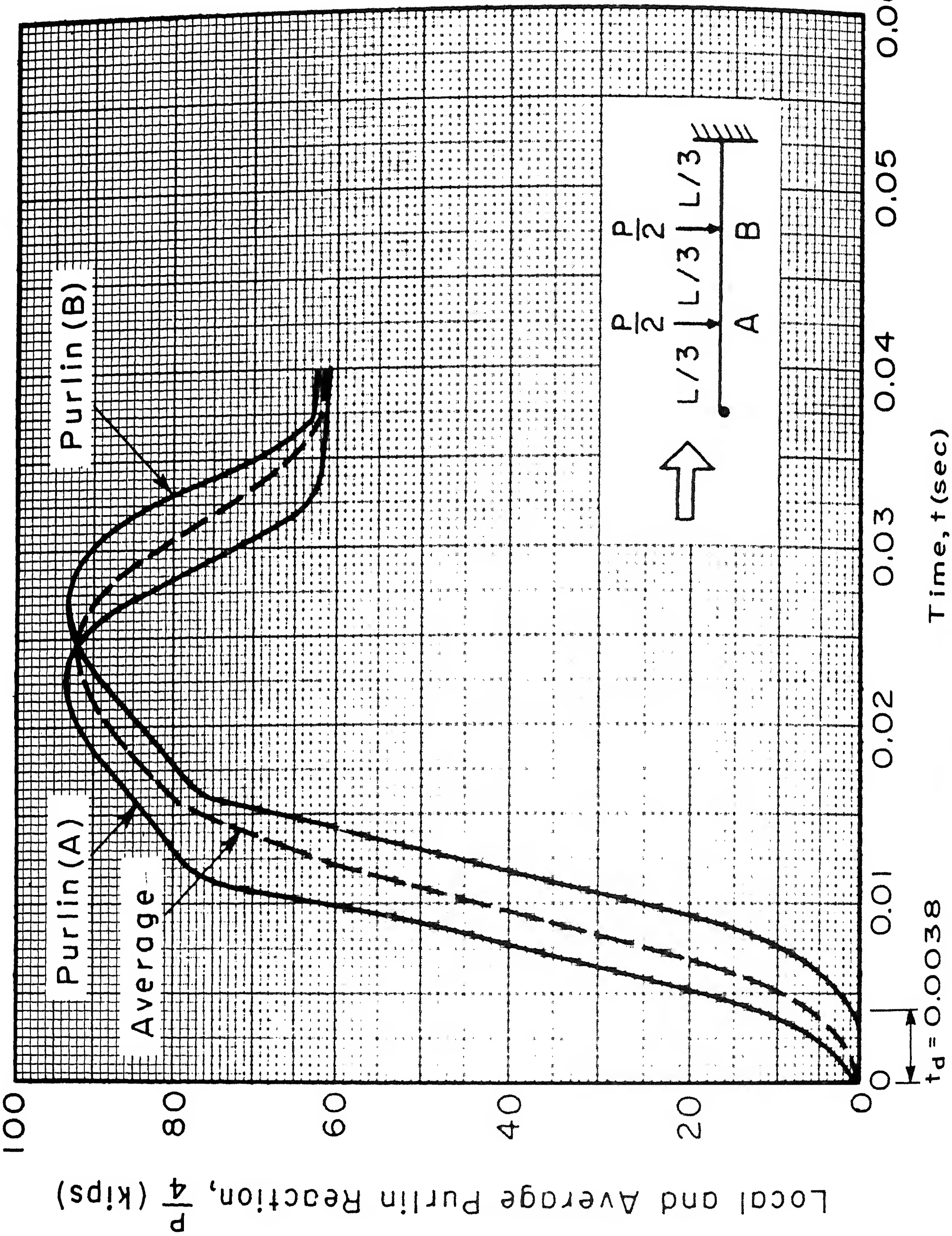


Figure 7.51. Girder design load

The resistance diagram is shown by figure 7.52.

Total concentrated mass = 0.40 kip-sec^2/ft

Total uniform mass = 0.075 kip-sec^2/ft

Concentrated load and mass, $K_{LM} = 0.83$ (table 6.1)

Concentrated load and uniform mass, $K_{LM} = 0.55$ (table 6.1)

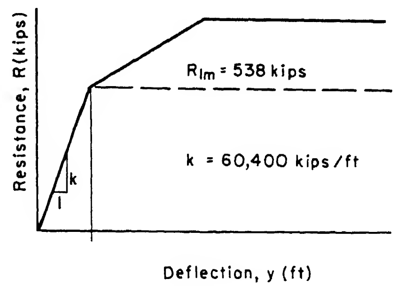


Figure 7.52. Resistance function for 36 WF 150 girder spanning 20 ft, fixed at one end and pinned at the other

$$T_n = 2\pi \left[\frac{\Sigma K_{LM}(m)}{k} \right]^{1/2} = 0.0156 \text{ sec}$$

Use $\Delta t = 0.0015$ sec $< \frac{T_n}{10} = 0.00156$ (par. 5-08)

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(0.0015)^2}{0.40(0.83) + 0.55(0.075)} = 6.01(10^{-6})(P_n - R_n) \text{ ft}$$

The numerical integration in table 7.21 is based on the equation $y_{n+1} = \ddot{y}_n(\Delta t)^2 + 2y_n - y_{n-1}$ (table 5.3). The dynamic reaction equations are:

$V_1 = 0.17R + 0.17P$ (table 6.1)

$V_2 = 0.33R + 0.33P$

The P_n values for the second column in table 7.21 are obtained from figure 7.51 multiplying by 4 to obtain the total concentrated load applied to the girder by the two purlins.

In table 7.21 the maximum R_n = 423 kips < 538 kips. The design is satisfactory.

b. Shear Stress Check.

V = 253 kips (table 7.21)

Multiply by 1.75 in accordance with paragraph 6-09

$$v = \frac{V}{dt_w} = \frac{253,000(1.75)}{38.84(0.625)} = 18,200 \text{ psi}$$

Allowable v = 21,000 psi; OK (par. 4-05c)

c. Check Proportions of 36 WF 150 for Local Buckling.

$b/t_f = 11.972/0.94 = 12.7 < 14.0$; OK

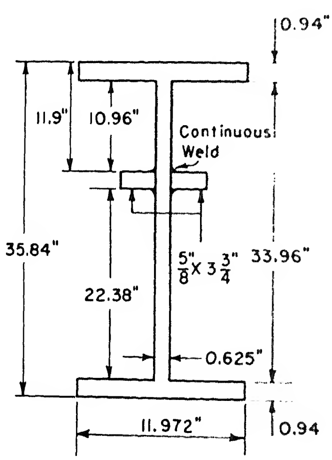
$a/t_w = 33.96/0.625 = 54.3 > 30$; NG

Longitudinal stiffeners are required (par. 4-06d)

7-35c

Table 7.21. Determination of Maximum Deflection and Dynamic Reactions for Roof Girder

t (sec)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$y_n(\Delta t)^2$ (ft)	y_n (ft)	V_n (kips)
0	0	0	0.6	0.000004	0	
0.0015	4	0.2	3.8	0.000023	0.000004	
0.0030	12	1.9	10.1	0.000061	0.000031	
0.0045	26	7	19	0.000114	0.000119	
0.0060	54	19	35	0.000210	0.000321	
0.0075	84	44	40	0.000240	0.000733	
0.0090	136	84	52	0.000313	0.001385	
0.0105	180	142	38	0.000228	0.002350	
0.0120	232	214	18	0.000108	0.003543	
0.0135	268	293	-25	-0.000150	0.004844	
0.0150	294	362	-68	-0.000409	0.005995	
0.0165	328	407	-79	-0.000475	0.006737	
0.0180	340	423	-83	-0.000499	0.007004	
0.0195	352	414	-62	-0.000373	0.006862	253 max
0.0210	360	383	-23	-0.000138	0.006347	
0.0225	366	344	+22	+0.000132	0.005694	
0.0240	372	312	60	0.000361	0.005173	
0.0255	370	303	67	0.000403	0.005013	
0.0270	366	317	49	0.000294	0.005256	
0.0285	352	350	2	0.000012	0.005793	
0.0300	330	383	-53	-0.000319	0.006342	
0.0315	312	397	-85	-0.000511	0.006572	
0.0330					0.006291	



$t_s = 5/8 = 0.625$

$b_s/t_s = 6$; $b_s = 6t_s = 6(5/8) = 3.75$ in.

Use 2 longitudinal stiffeners, 2 plates 5/8 in. by 5-3/4 in. - full length

$c/t_w = (11.9 - 0.94)/0.625$

$= 17.5 < 22$; OK

$e/t_w = (23.32 - 0.94)/0.625 = 35.6 < 40$; OK

Weight of added stiffener plates = 15.9 lb/ft

7-36 FINAL DESIGN OF COLUMN. The column design was begun in paragraph 7-34. The final steps in the column design are illustrated by this paragraph. The steps which follow all lead to the determination of the lateral deflection of the top of the columns by a numerical integration. In the preliminary design of the column (par. 7-34) some of the factors which affect the maximum deflection of the column are neglected to simplify the computations. These factors which are now considered are: the variation of plastic hinge moment with direct stress, the variation of column resistance with lateral deflection, the effect of girder flexibility on the stiffness of the columns, the difference between the load on the wall slab and the dynamic reactions from the wall which are used as the lateral design load for the frame columns.

For all pertinent dimensions refer to paragraph 7-34.

a. Mass Computation. (par. 7-34b)

Walls = 73.5 kips, roof slab = 57.3 kips, purlins = 7.2 kips

$$\text{Columns} = \frac{120(8.67)4}{1000} = 4.2 \text{ kips}$$

$$\text{Girders} = \frac{160(49)}{1000} = 7.9 \text{ kips}$$

Connections = 0.1(7.2 + 7.9) = 1.5 kips

Single-degree-of-freedom mass = total roof + 1/3 (columns + walls)

$$= \frac{57.3 + 7.2 + 7.9 + 1.5 + 0.33(73.5 + 4.2)}{32.2} = \frac{99.8}{32.2}$$

$= 3.1$ kip-sec^2/ft

b. Column Properties.

12 WF 120, I = 1071.7 in.^{4}, S = 163.4 in.^{3}, Z = 186.0 in.^{3}, 0.5(S + Z) = 174.7 in.^{3}, A = 35.31 in.^{2}, b = 12.32 in., t_f = 1.106 in., a = 10.91 in., t_w = 0.71 in., d = 13.12 in., r = 3.13 in.

c. Column Interaction Design Data. The plastic hinge moment, plastic axial load, and the values of M_1 and P_1 are computed below from the column properties.

$$M_P = f_{dy}(S + Z)0.5 = \frac{41.6}{12}(174.7) = 606 \text{ kip-ft} \quad (\text{eq } 4.2)$$

$$P_P = f_{dy}A = 41.6(35.31) = 1470 \text{ kips} \quad (\text{eq } 4.7)$$

$$P_1 = \frac{f_{dy}(2bt_f^{\,2} + t_w d^2/2 - 2t_w t_f^{\,2})}{d} \quad \text{(eq 4.12)}$$

$$= \frac{41.6}{13.12}\left[2(12.32)(1.106)^2 + 0.5(0.71)13.12^2 - 2(0.71)(1.106)^2\right]$$

$$= 288 \text{ kips}$$

$$M_1 = \frac{f_{dy}\left[4t_w(\frac{d}{2} - t_f)^3 + bt_f(3d^2 - 6dt_f + 4t_f^{\,2})\right]}{3d} \quad \text{(eq 4.11)}$$

$$M_1 = \frac{41.6}{12(3)(13.12)}\Big\{4(0.71)(5.46)^3 +$$

$$12.32(1.106)[3(13.12)^2 - 6(13.12)(1.106) + 4(1.106)^2]\Big\} = 555 \text{ kip-ft}$$

For $P_D > P_1$

$$M_D = \left(\frac{P_P - P_D}{P_P - P_1}\right)M_1 = \left(\frac{1470 - P_D}{1470 - 288}\right)555 = 690 - 0.469P_D \quad \text{(eq 4.13)}$$

For $P_D < P_1$

$$M_D = M_P - \frac{P_D}{P_1}(M_P - M_1) = 606 - \frac{606 - 555}{288}P_D = 606 - 0.177P_D \quad \text{(eq 4.14)}$$

d. <u>Effect of Girder Flexibility.</u> The relative flexibility of the girders reduces the spring constant k in the elastic range from the value obtained in paragraph 7-34 for the assumption of infinitely stiff girders (par. 7-08). (See par. 7-23d.) To obtain this revised value of k a simple sidesway analysis of the frame is performed. From the sidesway analysis the spring constant is the magnitude of the lateral force required to cause a unit displacement.

In figure 7.55 (page 14[illegible]) the initial spring constant is applicable only up to the formation of the first hinge. However, to simplify the design procedure, this value is assumed to hold up to the plastic resistance R_m.

In the sidesway analysis (fig. 7.5[illegible]) the frame dimensions are based on the centerline dimensions of the girders and columns. The lateral deflection for which the F.E.M. at top and bottom of each column is 1000 kip-ft is

$$x = (\text{F.E.M.})h^2/6EI$$

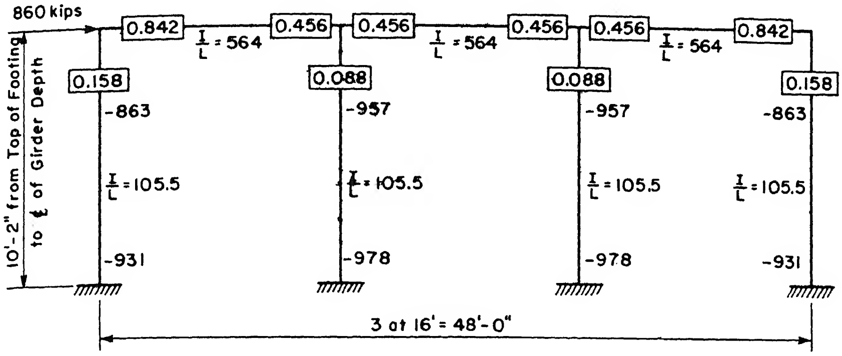


Figure 7.53. Sidesway frame analysis by moment distribution

$$x = \frac{(1000)(10.17)^2 144}{6(30)10^3(1071.7)} = 0.077 \text{ ft}$$

$$R = \frac{\Sigma M}{h} = \frac{2(863 + 957 + 931 + 978)}{10.17} = 734 \text{ kips}$$

$$k = \frac{R}{x} = \frac{734}{0.077} = 9540 \text{ kips/ft}$$

e. Loading. Both horizontal and vertical loads are considered. The lateral load for the numerical integration F_n (table 7.22, page 144) is obtained from the V_n dynamic reaction column of the numerical integration analysis of the front wall slabs in paragraph 7-31 (table 7.16). The dynamic reaction values for a 1-ft width are multiplied by 18, the width of one bay of the building. After the dynamic reactions of the front wall slab have decreased to the level of the applied load (at t = 0.018 sec) the F_n values are taken from the net lateral overpressure curve (fig. 7.43). For $\overline{P}_{net}$ in psi

$$F_n = \frac{144(18)6.18}{1000} \overline{P}_{net} = 16.0\overline{P}_{net} \text{ kips}$$

These data are plotted in figure 7.54 to give the frame lateral design load for the second column of table 7.22. The dimension 6.18 is equal to one-half the clear height of the wall plus the roof slab thickness.

$$\frac{11.67}{2} + 0.35 = 6.18 \text{ ft}$$

The total vertical load P_n in table 7.22 is obtained by multiplying the average roof overpressure (fig. 7.44) by [54(18)144]/1000 = 140 and

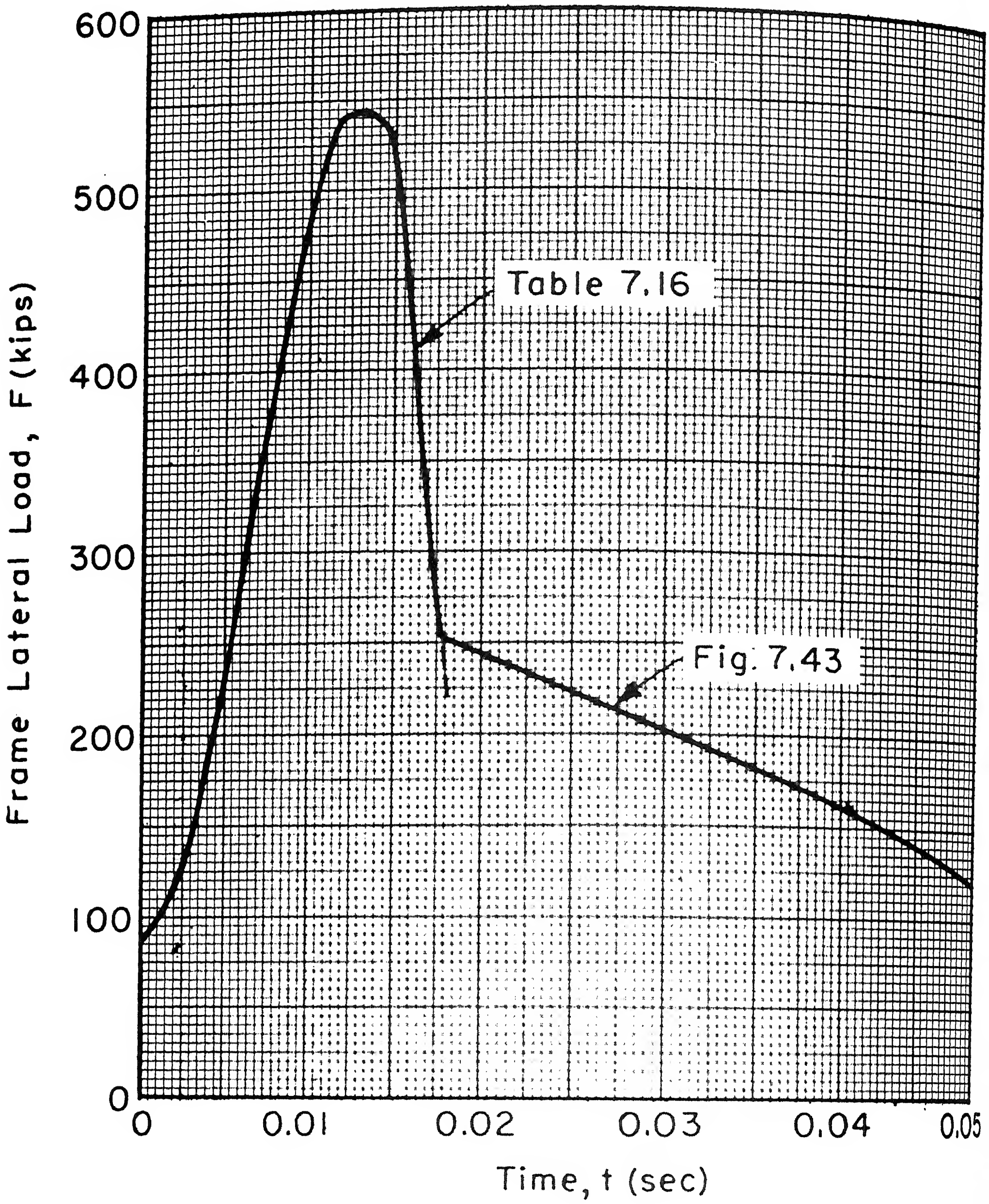


Figure 7.54. Frame design lateral load

adding to that the dead weight of the roof system 57.3 + 7.2 + 7.9 + 1.5 = 73.9 kips. The average roof overpressure is used because in the numerical analysis only the average column load is needed. This procedure neglects the dynamic effect of the roof slab, purlins, and girder on the vertical blast loads.

7-36f

f. Computation of Deflection of Frame by Numerical Integration. (See table 7.22.) The $(P_D)_n$ values are the average axial column loads obtained by dividing the total vertical load P_n by the number of columns. The $(P_D)_n$ values are used to determine the corresponding values of $(M_D)_n$ by the relationships of paragraph 7-36c. From the value of $(M_D)_n$ the maximum resistance is obtained by the relation

$$(R_m)_n = \frac{2n(M_D)_n}{h_c} = \frac{2(4)(M_D)_n}{8.67} = 0.922(M_D)_n$$

R_n is equal to $kx_n = 9540x_n$ in the elastic range. The expression $\frac{P_n x_n}{h_c}$ indicates the decrease in resistance corresponding to the increase in bending moment resulting from the eccentric loading.

The basic equation for the numerical integration in table 7.22 is

$$x_{n+1} = \ddot{x}_n(\Delta t)^2 + 2x_n - x_{n-1} \quad \text{(table 5.3)}$$

where

$$\ddot{x}_n(\Delta t)^2 = \left[F_n - R_n + \frac{P_n x_n}{h_c}\right]\frac{(\Delta t)^2}{m} = \left[F_n - R_n + \frac{P_n x_n}{h_c}\right]\frac{(0.002)^2}{3.1}$$

$$= 1.29(10^{-6})\left[F_n - R_n + \frac{P_n x_n}{h_c}\right] \text{ ft}$$

To check the slenderness critéria (par. 4-08) the following equation is evaluated for each time interval

$$\frac{M_D}{M_P}\left[\frac{K'Ld}{100bt_f}\right] + \frac{P_D}{P_P}\left[\frac{K''L}{15r}\right] \leqslant 1 \quad \text{(eq 4.10)}$$

$K' = 0.14$ (table 4.1), $K'' = 0.50$ (table 4.2)

$L = 8.67$ ft (clear height), $b = 12.32$ in., $r = 3.13$ in.

$t_f = 1.106$ in., $d = 13.12$ in.

$$\frac{M_D}{M_P}\left[\frac{(0.14)(8.67)12(13.12)}{100(12.32)1.106}\right] + \frac{P_D}{P_P}\left[\frac{0.50(8.67)12}{15(3.13)}\right] \leqslant 1$$

$$(0.140)\frac{M_D}{M_P} + (1.11)\frac{P_D}{P_P} \leqslant 1$$

The time interval $\Delta t = 0.002$ sec used in table 7.22 is based on the natural period $T_n = 2\pi\sqrt{m/k} = 6.28\sqrt{3.1/9540} = 0.113$ sec.

$$\Delta t \approx \frac{T_n}{10} = \frac{0.113}{10} = 0.0113 \text{ (par. 5-08)}$$

The analysis in table 7.22 shows that the maximum deflection

Table 7.22. Determination of Column Adequacy

t (sec)	P_n (kips)	$(P_D)_n$ (kips)	M_D (kip-ft)	R_m (kips)	F_n (kips)	R_n (kips)	$\frac{\Sigma P_n}{h_c} x_n$ (kips)	$\frac{\Sigma P_n}{h_c}$	Σ	$\ddot{x}_n (\Delta t)^2$ (ft)	x_n (ft)
0	74	18	603	556	85	0	0	8.5	42.5	0.000055	0
0.002	136	34	600	554	115	0.5	0	15.7	114.5	0.000148	0.000055
0.004	199	50	597	551	185	2.5	0	23.0	182.5	0.000235	0.000258
0.006	261	65	594	548	285	6.6	0	30.1	278.4	0.000359	0.000696
0.008	323	81	592	546	390	14.2	0.1	37.3	375.9	0.000485	0.001493
0.010	386	96	589	543	435	26.5	0.1	44.5	458.6	0.000591	0.002775
0.012	448	112	586	541	543	44.3	0.2	51.7	498.9	0.000644	0.004648
0.014	511	128	583	538	540	69.4	0.4	58.9	472.0	0.000609	0.007165
0.016	573	143	581	536	420	98.2	0.7	66.1	322.5	0.000416	0.010291
0.018	635	159	578	533	252	132.0	1.0	73.2	121.0	0.000156	0.013833
0.020	[illegible]	[illegible]	[illegible]	531	245	167.2	1.4	80.5	79.2	0.000102	0.017531
0.022	[illegible]	[illegible]	[illegible]	[illegible]	235	203.5	1.9	87.6	33.4	0.000043	0.021331
0.024	803	[illegible]	[illegible]	[illegible]	227	240.2	2.4	94.9	-10.8	-0.000014	0.025174
0.026	[illegible]	[illegible]	[illegible]	[illegible]	220	276.7	3.0	102.1	-53.7	-0.000069	0.029003
0.028	[illegible]	[illegible]	[illegible]	[illegible]	212	312.6	3.6	109.2	-99.0	-0.000128	0.032763
0.030	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	347.2	4.2	116.5	-141.0	-0.000182	0.036395
0.032	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	377.1	4.9	123.6	-179.2	-0.000231	0.039845
0.034	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	5.6	130.8	-217.2	-0.000280	0.043064
0.036	[illegible]	[illegible]	[illegible]	[illegible]	180	435.9	6.4	138.1	-252.5	-0.000326	0.046003
0.038	[illegible]	[illegible]	[illegible]	[illegible]	172	463.8	7.0	145.2	-284.8	-0.000367	0.048616
0.040	[illegible]	317	[illegible]	[illegible]	164	485.2	7.4	146.3	-313.8	-0.000405	0.050862
0.042	1261	315	542	500	156	500.0	7.7	145.4	-336.3	-0.000434	0.052703
0.044	1254	313	543	501	148	501.0	7.8	144.6	-345.2	-0.000445	0.054110
0.046	1244	311	544	502	140	502.0	7.9	143.4	-354.1	-0.000457	0.055072
0.048	1233	308	546	504	132	504.0	7.9	142.2	-364.1	-0.000470	0.055577
0.050	1222	306	547	505	124	505	7.8	140.9	-373.2	-0.000481	0.055612 *
											0.055166

* $(x_n)_{max}$ = 0.056 ft.

7-36g

$(x_n)_{max} = 0.056$ exceeds the limiting elastic deflection $x_e = \frac{R_m}{9540} = \frac{500}{9540} = 0.053$. This small amount of plastic action is considered acceptable for purposes of this example. At t = 0.04 sec the slenderness criteria equation

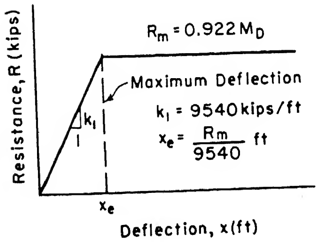


Figure 7.55. Frame resistance diagram

$$0.140 \frac{M_P}{M_P} + 1.11 \frac{P_P}{P_P} = 0.14 \frac{541}{606} + 1.11 \frac{317}{1470}$$

$$= 0.364 < 1.0, \therefore \text{OK}$$

g. Shear Stress Check.

$(R_m)_{max} = 505$ kips (table 7.22)

$V_{max} = 505/4 = 126.0$ kips

$$v = \frac{V}{dt_w} = \frac{126{,}000}{(13.12)0.71} = 13{,}400 \text{ psi}$$

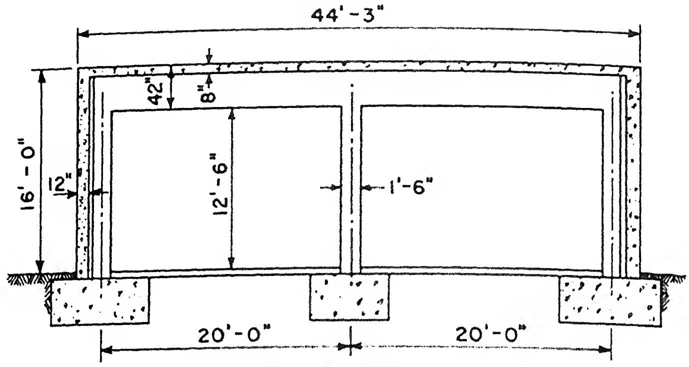
Allowable v = 21,000 psi; OK (par. 4-05e)

NUMERICAL EXAMPLE, DESIGN OF A ONE-STORY REINFORCED-CONCRETE FRAME BUILDING, PLASTIC DEFORMATION PERMITTED

7-37 GENERAL. This numerical example presents the design of the important elements of one bay of a windowless one-story, reinforced-concrete frame building. Included are the design of the wall slab, roof slab, roof girder, and columns. The foundation design is not included because that design procedure is illustrated by paragraph 7-27.

In this example all elements with the exception of the girder are permitted to deform plastically. The reason for designing the girder in the elastic range is covered in paragraph 7-11. The limiting deflections are established on the basis of paragraph 6-26.

The structure consists of reinforced-concrete wall slabs spanning vertically with one-way reinforcement. The wall slab is supported at the bottom by the foundation and at the top by the roof slab. The roof slab is also a one-way slab spanning continuously over the reinforced concrete frames. The roof girders are tee beams formed by the roof slab and rectangular girder stems. The columns are rectangular tied columns symmetrically reinforced in the strong direction.



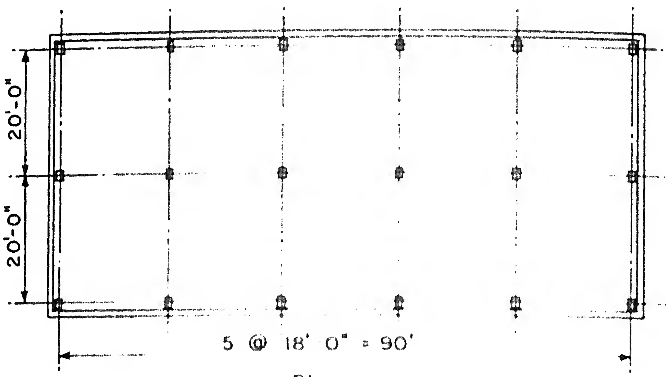


Figure 7.56. Plan and section of reinforced concrete building

7-38 DESIGN PROCEDURE. The design procedure illustrated by this example is essentially the same as the procedure detailed in paragraph 7-18 for the plastic design of a steel frame building with reinforced-concrete walls and roof.

7-39 LOAD DETERMINATION. In this example only the completed overpressure curves are presented. The computation of the overpressure variation is explained in detail in EM 1110-345-413 and illustrated again in paragraph 7-19. The design overpressure of 10 psi is selected arbitrarily.

The overpressure vs time curves that are presented are:

(1) Incident overpressure vs time (fig. 7.57)

(2) Front face overpressure vs time (fig. 7.58)

(3) Rear face overpressure vs time (fig. 7.59)

(4) Net lateral overpressure vs time (fig. 7.60)

(5) Average roof overpressure vs time (fig. 7.61)

7-39

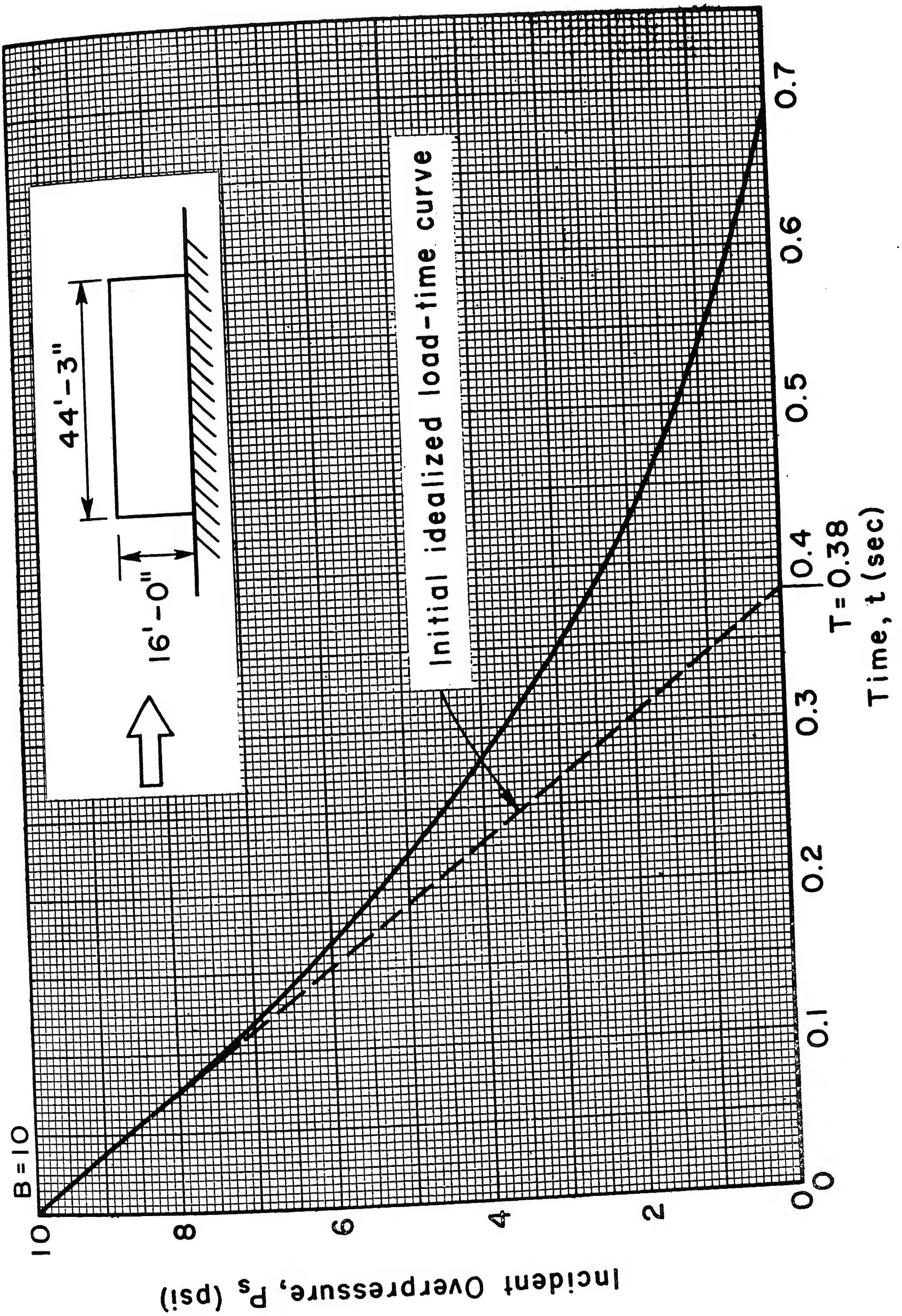


Figure 7.57. Incident overpressure vs time curve

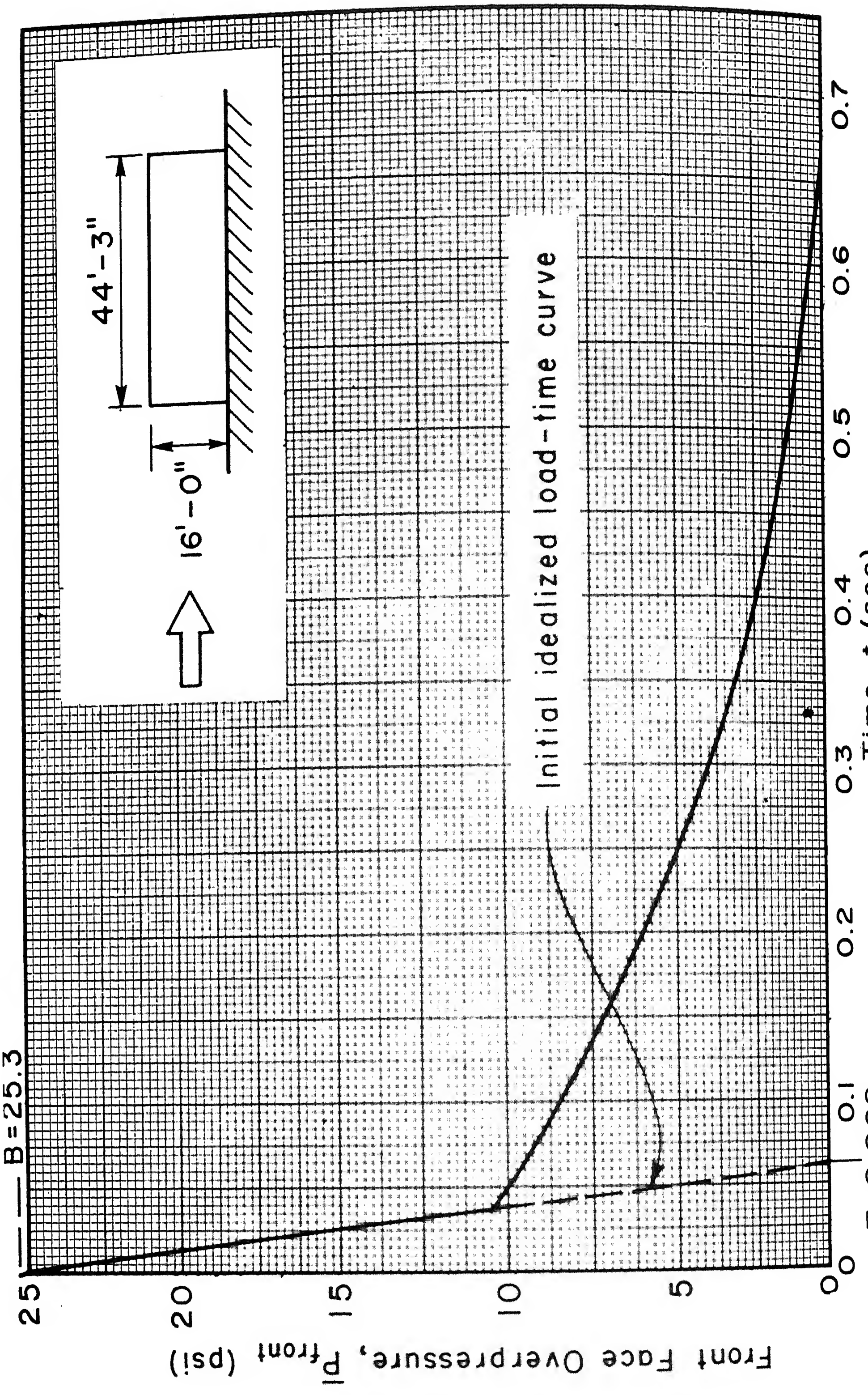


Figure 7.58. Front face overpressure vs time curve

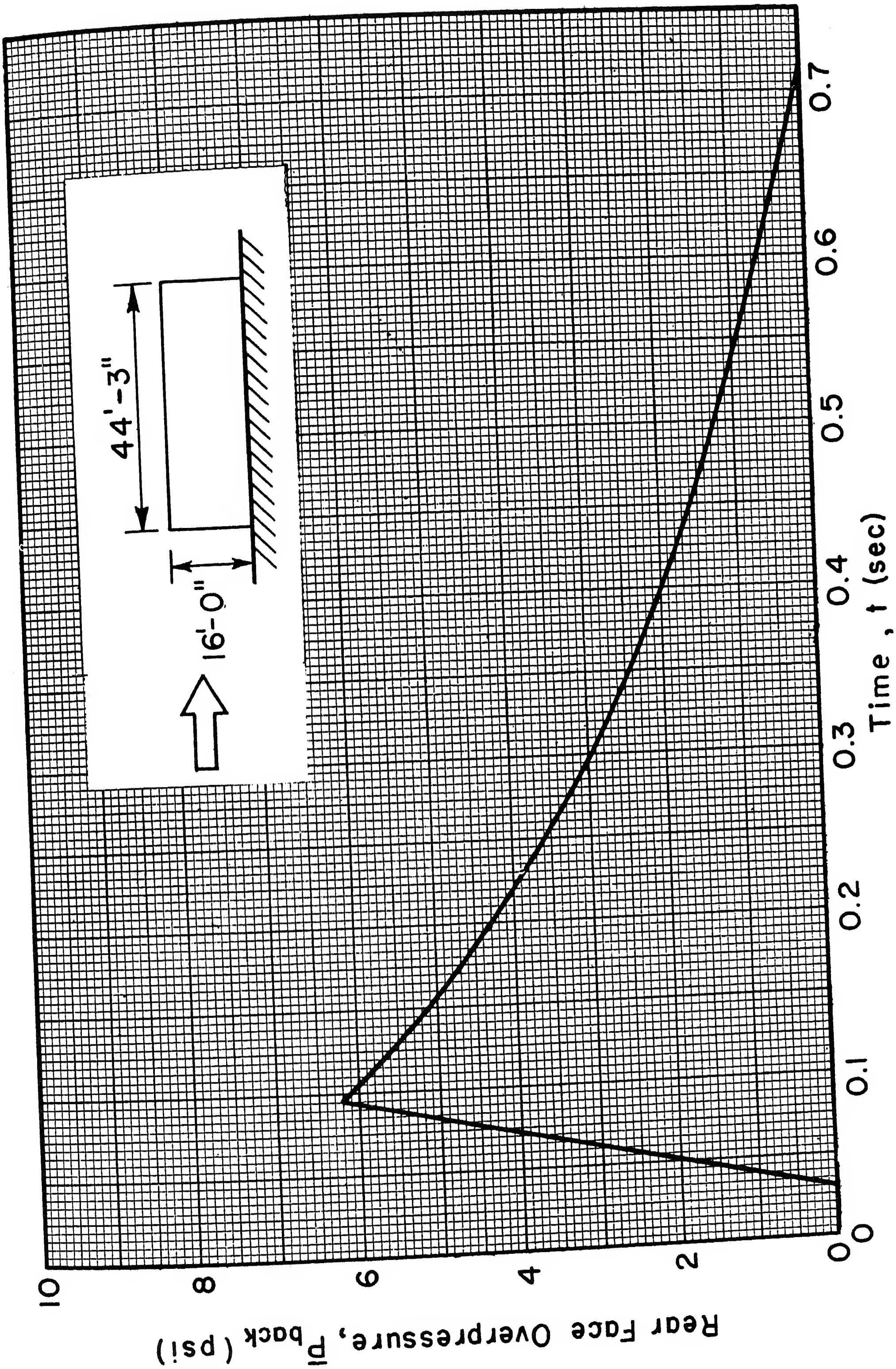


Figure 7.59. Rear face overpressure vs time curve

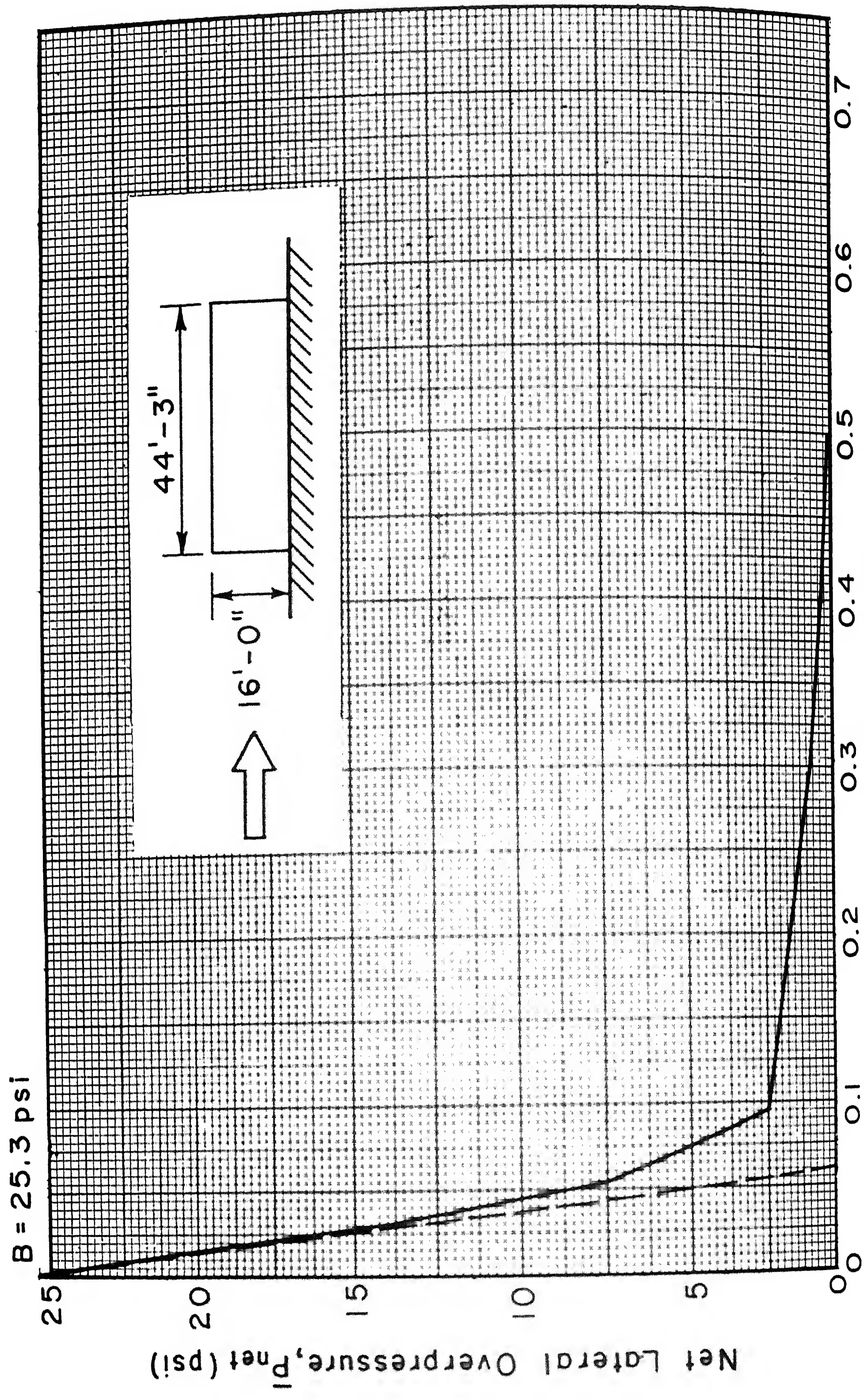


Figure 7.60. Net lateral overpressure vs time curve

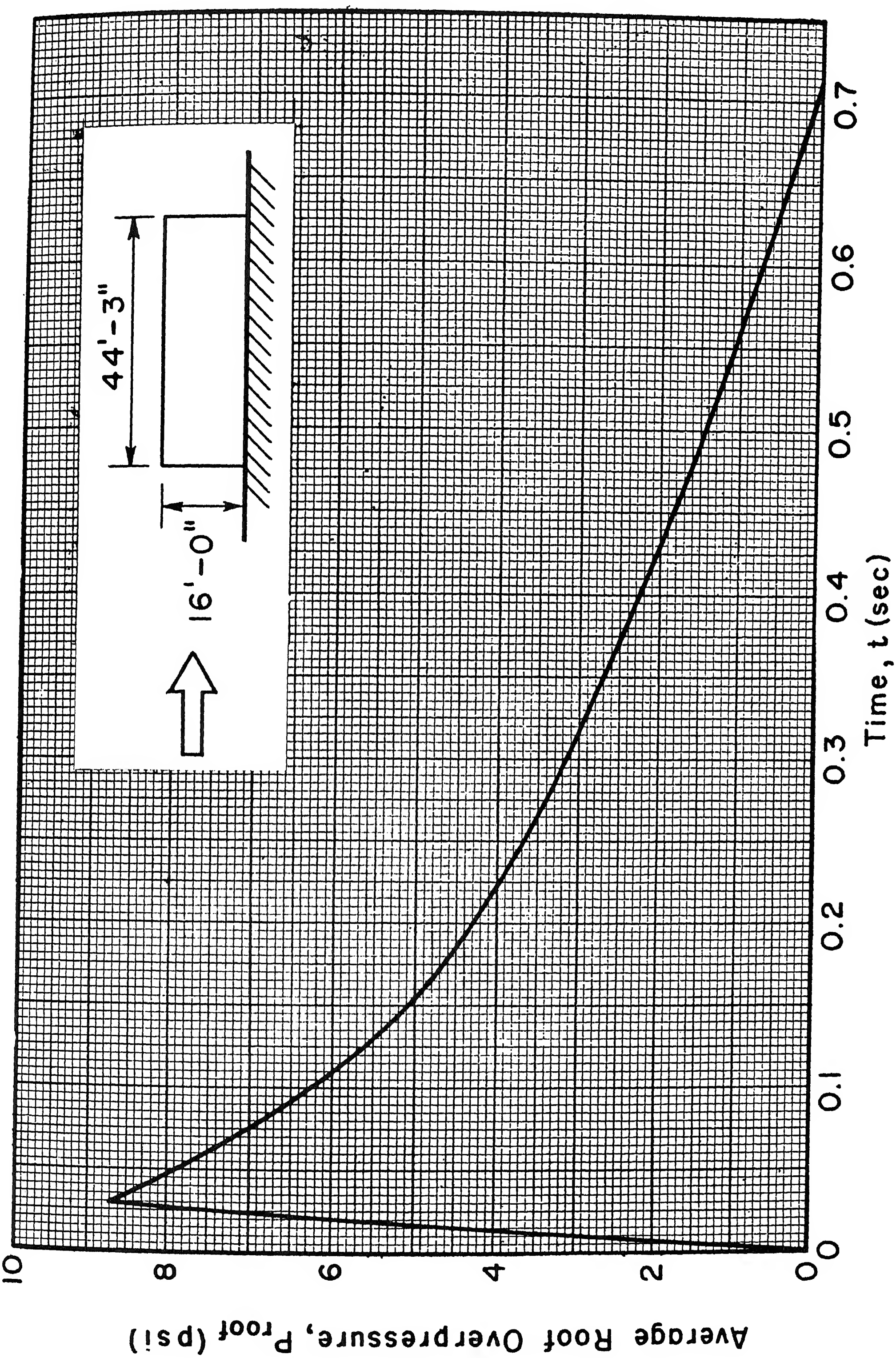
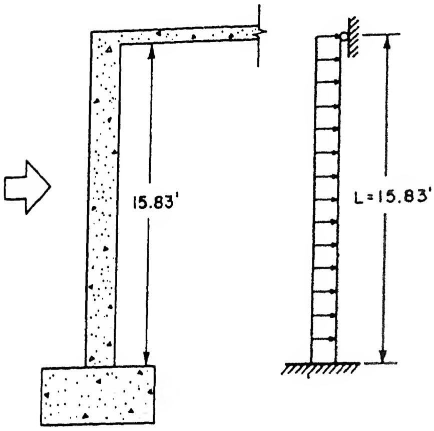


Figure 7.61. Average roof overpressure vs time curve

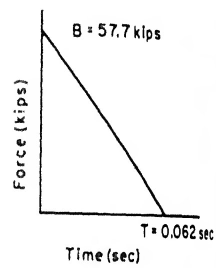
7-40 DESIGN OF WALL SLAB. The wall slab is designed as a reinforced concrete beam one foot in width fixed at the foundation and pinned at the roof where it is framed into a thin slab. The wall slab is designed for plastic action so that a plastic hinge will be developed first at the foundation and then at mid-span when the slab is subjected to the design loading. The clear height of the slab is considered the design span length.



The preliminary design procedure for plastic design is described and illustrated by an example in paragraph 6-11. The dead load stresses are neglected for a vertical wall slab.

a. Design Loading. The design load as idealized from the computed loading shown by figure 7.58 is defined by:

$$B = \frac{25.3(144)15.83}{1000} = 57.7 \text{ kips}$$



$$T = 0.062 \text{ sec}$$

$$H = \frac{BT}{2} = \frac{(57.7)0.062}{2} = 1.79 \text{ kip-sec (par. 6-11)}$$

b. Dynamic Design Factors. (Refer to table 6.1.)

Elastic range:

$K_L = 0.58$, $K_M = 0.45$, $K_{LM} = 0.78$

$R_{1m} = \dfrac{8M_{Ps}}{L}$, $k_1 = \dfrac{185EI}{L^3}$

$V_1 = 0.26R + 0.12P$, $V_2 = 0.43R + 0.19P$

Elasto-plastic range:

$K_L = 0.64$, $K_M = 0.50$, $K_{LM} = 0.78$

$R_m = \dfrac{4}{L}(M_{Ps} + M_{Pm})$, $k_{ep} = \dfrac{384EI}{5L^3}$

$V = 0.39R + 0.11P$

Plastic range:

$K_L = 0.50,$ $K_M = 0.33,$ $K_{LM} = 0.66$

$$R_m = \frac{4}{L}(M_{Ps} + 2M_{Pm})$$

$$V = 0.38R + 0.12P$$

Average values:

$$K_L = \frac{(0.64 + 0.50)}{2} = 0.57$$

$$K_M = \frac{(0.50 + 0.33)}{2} = 0.42$$

$$R_m = \frac{4}{L}(M_{Ps} + 2M_{Pm})$$

$$k_E = \frac{160EI}{L^3} \quad (\text{if } M_{Ps} = M_{Pm})$$

c. First Trial - Actual Properties.

Let $M_{Ps} = M_{Pm} = M_P$

Assume $p = 0.015$ (par. 4-10)

Let $\alpha\beta = 5$ (par. 6-26)

Assume $C_R = 0.75$ (experience)

$R_m = C_R B = 0.75(57.7) = 43.3$ kips

(eq 4.16)

$$M_P = pf_{dy}bd^2\left(1 - \frac{pf_{dy}}{1.7f'_{dc}}\right)$$

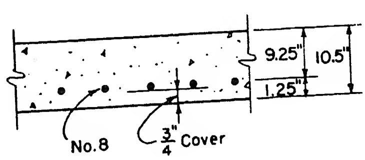
$$= 0.015(52)(1)d^2\left[1 - \frac{0.015(52)}{1.7(3.9)}\right] = 0.688d^2 \text{ kip-ft (d in inches)}$$

$$R_m = \frac{12M_P}{L} = \frac{(12)0.688d^2}{15.83} = 43.3, \therefore d = 9.11 \text{ in.}$$

Try $h = 10.5$ in., $d = 9.25$ in., $p = 0.015$, $np = 0.15$

$$M_P = 0.688(9.25)^2 = 58.9 \text{ kip-ft}$$

$$R_m = \frac{12M_P}{L} = \frac{12(58.9)}{15.83} = 44.6 \text{ kips}$$



$$I_g = bh^3/12 = (10.5)^3 = 1158 \text{ in.}^4$$

$$I_t = bd^3\left[\frac{k^3}{3} + np(1 - k)^2\right] = 12(d)^3\left[\frac{(0.42)^3}{3} + 0.15(1 - 0.42)^2\right]$$

$$= 0.905d^3 = 0.905(9.25)^3 = 716 \text{ in.}^4$$

$$I_a = 0.5(I_g + I_t) = 0.5(1158 + 716) = 937 \text{ in.}^4$$

$$k_E = \frac{160EI}{L^3} = \frac{(160)3(10)^3 937}{144(15.83)^3} = 787 \text{ kips/ft}$$

$$y_E = \frac{R_m}{k_E} = \frac{44.6}{787} = 0.0567 \text{ ft}$$

$$y_m = \alpha\beta\, y_E = 5(0.0567) = 0.2835 \text{ ft (par. 6-26)}$$

$$\text{Weight} = \frac{10.5(150)15.83}{(12)1000} = 2.078 \text{ kips}$$

$$\text{Mass } m = \frac{2.078}{32.2} = 0.0645 \text{ kip-sec}^2\text{/ft}$$

d. <u>First Trial - Equivalent System Properties.</u>

$$R_{me} = K_L R_m = 0.57(44.6) = 25.4 \text{ kips (eq 6.12)}$$

$$H_e = K_L H = 0.57(1.79) = 1.02 \text{ kip-sec (eq 6.8)}$$

$$m_e = K_M m = 0.42(0.0645) = 0.0271 \text{ kip-sec}^2\text{/ft (eq 6.2)}$$

$$W_p = \frac{(H_e)^2}{2m_e} = \frac{(1.02)^2}{2(0.0271)} = 19.20 \text{ ft-kips (eq 6.10)}$$

$$T_n = 2\pi\sqrt{\frac{K_{LM}m}{k_E}} = 6.28\sqrt{0.78(0.0645)/787} = 0.050 \text{ sec (eq 6.14)}$$

e. <u>First Trial - Work Done vs Energy Absorption Capacity.</u>

$$C_T = T/T_n = 0.062/0.050 = 1.24$$

$$C_R = R_m/B = 44.6/57.7 = 0.77 \text{ (eqs 6.15, 6.16)}$$

$$t_m/T = 0.71 \text{ (fig. 5.29)}$$

$$t_m = (0.71)0.062 = 0.044 \text{ sec}$$

The idealized load-time variation is considered to be a satisfactory approximation to the actual front face overpressure curve (fig. 7.58) up to $t_m = 0.044$ sec (par. 5-13).

$C_W = 0.31$ (fig. 5.27)

$W_m = C_W W_P = 0.31(19.20) = 5.95$ ft-kips (eq 6.17)

$E = R_{me}(y_m - 0.5y_E) = 25.4\ [0.2835 - 0.5(0.0567)]$

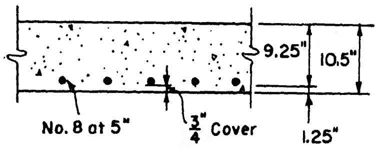
$= 6.48$ ft-kips (eq 6.18)

$E > W$, therefore the selected proportions are satisfactory as a preliminary design.

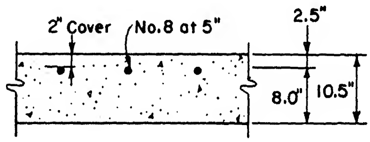
A 10-in. slab was tried and found to be unsatisfactory.

Use 10.5-in. slab.

f. <u>Preliminary Design for Bond Stress.</u> It is now necessary to select the reinforcing steel for the critical cross sections. At the fixed end of the wall the cover requirement results in a smaller value of d = 8.0 in. than at midspan, d = 9.25 in. To achieve approximately the same p at both critical cross sections several values of p are investigated to obtained the value of p for which



Midspan and Pinned End



Fixed End

$$\frac{8M_{Pm} + 4M_{Ps}}{12} = M_P = 58.9 \text{ kip-ft}$$

plot of equation (4.16) simplifies this computation. From such a plot ≈ 0.017.

<u>t the fixed end</u>

Estimated $V_{max} = 0.5R_m = 0.5(44.6) = 22.3$ kips

Allowable $u = 0.15f'_c = 0.15(3000) = 450$ psi

$$\Sigma o = \frac{V}{ujd} = \frac{8(22,300)}{7(450)8.0} = 7.1 \text{ in.}$$

$A_s = pbd = 0.017(12)8.0 = 1.63$ in.^{2}

Try #8 at 5 in., $A_s = 1.90$ in.^{2}, $\Sigma o = 7.5$ in.

$$p = \frac{1.90}{12(8)} = 0.0198$$

<u>t the pinned end</u>

$$\text{Estimated } V_{max} = 1/3R_m = \frac{44.6}{3} = 14.9 \text{ kips}$$

$$\Sigma o = \frac{V}{ujd} = \frac{8(14,900)}{450(7)9.25} = 4.1 \text{ in.}$$

$$A_s = pbd = 0.017(12)9.25 = 1.89 \text{ in.}^2$$

Try #8 at 5 in., $A_s = 1.9$ in.^{2}, $\Sigma o = 7.5$ in.

$$p = \frac{1.9}{12(9.25)} = 0.0171$$

g. Determination of Maximum Deflection and Dynamic Reactions by Numerical Integration.

$$M_{Pm} = pf_{dy}bd^2\left[1 - \frac{pf_{dy}}{1.7f'_{dc}}\right] = 0.0171(52)(1)(9.25)^2\left[1 = \frac{0.0171(52)}{1.7(3.9)}\right]$$

$$= 66.0 \text{ kip-ft (eq 4.16)}$$

$$M_{Ps} = 0.0198(52)(1)(8.0)^2\left[1 - \frac{0.0198(52)}{1.7(3.9)}\right] = 56.0 \text{ kip-ft}$$

$$I_g = bh^3/12 = (10.5)^3 = 1158 \text{ in.}^4$$

$$I_t = bd^3\left[\frac{k^3}{3} + np(1 - k)^2\right] = 12(9.25)^3\left[\frac{(0.44)^3}{3} + 0.171(1 - 0.44)^2\right]$$

$$= 775 \text{ in.}^4$$

$$I_a = 0.5(I_g + I_t) = 0.5(1158 + 775) = 967 \text{ in.}^4$$

$$\text{Weight} = \frac{10.5(150)15.83}{12(1000)} = 2.078 \text{ kips}$$

$$\text{Mass } m = \frac{2.078}{32.2} = 0.0645 \text{ kip-sec}^2\text{/ft}$$

Elastic range:

$$R_{1m} = \frac{8M_{Ps}}{L} = \frac{8(56.0)}{15.83} = 28.2 \text{ kips}$$

$$k_1 = \frac{185EI}{L^3} = \frac{(185)3(10)^3 967}{144(15.83)^3} = 930 \text{ kips/ft}$$

$$y_e = \frac{R_{1m}}{k_1} = \frac{28.2}{930} = 0.0304 \text{ ft}$$

Elasto-plastic range:

$$R_m = \frac{4(M_{Ps} + 2M_{Pm})}{L} = \frac{4[56.0 + 2(66.0)]}{15.83} = 47.5 \text{ kips}$$

$$k_{ep} = \frac{384EI}{5L^3} = \frac{384}{5(185)} k_1 = 386 \text{ kips/ft}$$

$$y_{ep} = y_e + \frac{R_m - R_{1m}}{k_{ep}} = 0.0304 + \frac{47.5 - 28.2}{386} = 0.080 \text{ ft}$$

Plastic range:

$R_m = 47.5$ kips

By providing equal areas under the "effective resistance" line and he computed elasto-plastic resistance line, $Y_E = 0.0641$ ft (fig. 7.62).

$$k_E = R_m/y_E = 47.5/0.0641 = 740 \text{ kips/ft}$$

$$T_n = 2\pi\sqrt{K_{LM}(m)/k_E}$$

$$= 6.28\sqrt{\frac{0.78(0.0645)}{740}} = 0.0517 \text{ sec}$$

$$y_m = \alpha\beta\, y_E = 5(0.0641) = 0.3205 \text{ ft}$$

The basic equation for the numerical ntegration in table 7.23 is $y_{n+1} = \ddot{y}_n(\Delta t)^2 + 2y_n - y_{n-1}$ (table 5.3) where

$$\ddot{y}_n(\Delta t)^2 = \frac{P_n - R_n(\Delta t)^2}{K_{LM}(m)}$$

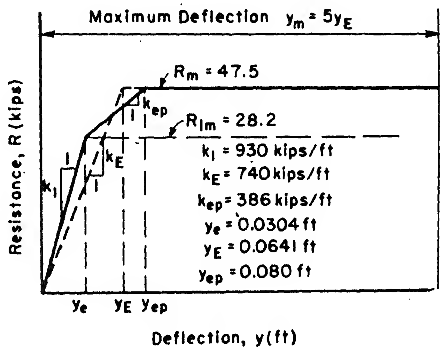


Figure 7.62. Resistance function for 10-1/2-in. slab spanning 15.83 ft, fixed at one end and pinned at the other

Table 7.23. Determination of Maximum Deflection and Dynamic Reactions for Front Wall Slab

t (sec)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)	V_{1n} (kips)	V_{2n} (kips)
0	57.7	0	28.8	0.0143	0	6.9	11.0
0.005	53.0	13.3	39.7	0.0197	0.0143	9.8	15.8
0.010	48.3	35.1	13.2	0.00656	0.0483	19.0	19.0
0.015	43.7	47.5	- 3.8	-0.00223	0.0888	23.2	23.2
0.020	39.1	47.5	- 8.4	-0.00493	0.1271	22.7	22.7
0.025	34.3	47.5	-13.2	-0.00775	0.1605	22.1	22.1
0.030	29.7	47.5	-17.8	-0.01045	0.1862	21.6	21.6
0.035	26.4	47.5	-21.1	-0.01239	0.2015	21.2	21.2
0.040	25.3	47.5	-22.2	-0.01303	0.2044*	21.2	21.2
0.045	24.2	38.1			0.1943	12.8	18.5

$*(y_n)_{max} = 0.20$ ft.

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)25(10)^{-6}}{0.78(0.0645)} = 4.97(10)^{-4}(P_n - R_n) \text{ ft, elastic range}$$

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)25(10)^{-6}}{0.78(0.0645)} = 4.97(10)^{-4}(P_n - R_n) \text{ ft, elasto-plastic range}$$

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)25(10)^{-6}}{0.66(0.0645)} = 5.87(10)^{-4}(P_n - R_n) \text{ ft, plastic range}$$

The time interval $\Delta t = 0.005$ sec is approximately $T_n/10 = 0.00517$ sec (par. 5-08).

The dynamic reaction equations are listed in paragraph 7-40b. The P_n values for the second column are obtained from figure 7.58, multiplying by $144(15.83)/1000 = 2.28$.

The maximum deflection $(y_n)_{max}$, computed in table 7.23, is 0.204 ft, which is less than the allowable y_m of 0.3205 ft. Thus

$$\alpha\beta = \frac{0.204}{0.3205}(5) = 3.2; \text{ OK}$$

h. Shear and Bond Strength. For bottom of wall (fixed end of idealized slab):

$V_{max} = 23.0$ kips (table 7.23)

For no shear reinforcement, allowable $v_y = 0.04f'_c + 5000p$ (eq 4.24)

$$v_y = 0.04(3000) + 5000(0.0171) = 120 + 85 = 205 \text{ psi}$$

$$v = \frac{8V}{7bd} = \frac{8(23,200)}{7(12)8.0} = 274 \text{ psi}$$

Shear reinforcement required for $274 - 205 = 69$ psi

Contribution of shear reinforcement to allowable shear stress $= rf_y$

$$r = \frac{69}{40,000} = 0.0017$$

Try 1 #3, $A_s = 0.11$ in.^{2}

$$r = \frac{A_s}{bs} = \frac{0.11}{10(s)} = 0.0017; \therefore s = 6.5 \text{ in., use } s = 6 \text{ in.}$$

For top of wall (pinned end of idealized slab)

$V_{max} = 23.2$ kips (table 7.23)

$v_y = 205$ psi

$$v = \frac{8V}{7bd} = \frac{8(23,200)}{7(12)9.25} = 240 \text{ psi}$$

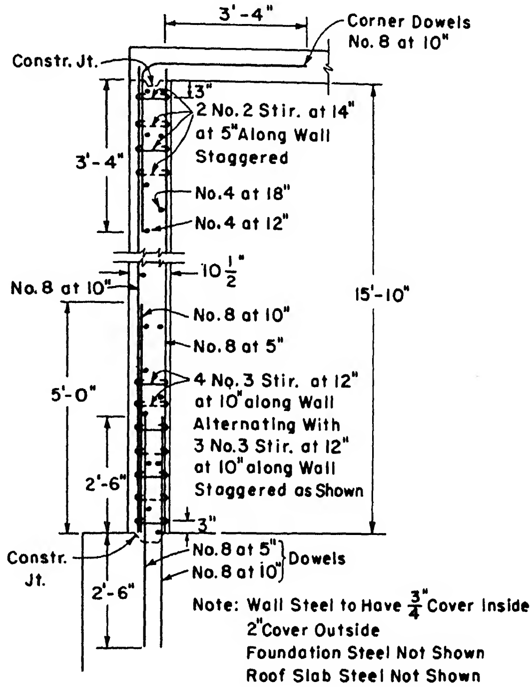
Shear reinforcement required for 240 - 205 = 35 psi

$$r = 35/40,000 = 0.00088$$

Try 1 #2, $A_s = 0.05$ in.^{2}

$$r = \frac{A_s}{bs} = \frac{0.05}{10(s)} = 0.00088;$$

$$\therefore s = 5.7 \text{ in., use } s = 5 \text{ in.}$$



Bond:

Allowable $u = 0.15f'_c = 0.15(3000)$

$= 450$ psi

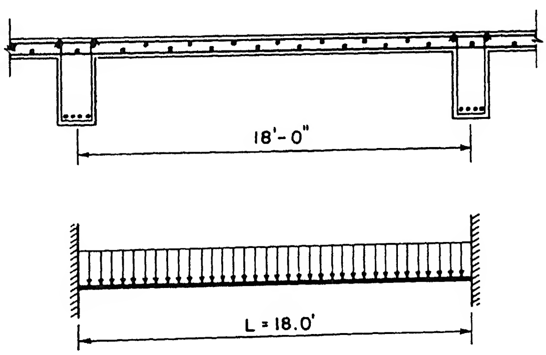
$$\Sigma o = \frac{V}{ujd} = \frac{8(23,000)}{7(450)(8.0)} = 7.3 \text{ in.}$$

Use #8 at 5 in.,

$\Sigma o = 7.5$ in., $A_s = 1.9$ in.^{2}

i. Summary. 10-1/2-in. slab (see sketch above)

7-41 DESIGN OF ROOF SLAB. The roof is formed by a one-way slab spanning continuously over the frame bents spaced at 18 ft. This manual is limited to consideration of single-span elements. To account for the continuity of the slab it is designed as a fixed-end beam of 18-ft span. Since plastic behavior is desired the design procedure of paragraph 6-11 is followed. The permissible deflection is established in accordance with paragraph 6-26 and the static loads are allowed for by reducing the resistance.



a. Design Loading. The roof slab spanning between frames in a long building has different design conditions from the roof framing over purlins in a similar building (par. 7-22).

For the blast wave moving parallel to the long axis of the building the overpressure variation with time at all points is the same. However, for any slab element the overpressure along the span varies with time as a result of the lag time

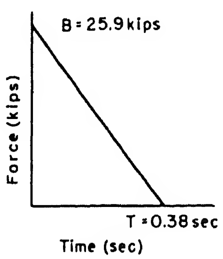
$$t_{lag} = \frac{\text{length of slab}}{\text{velocity of blast wave}} = \frac{18}{1403} = 0.0128 \text{ sec}$$

For this case the average slab loading is determined by introducing the rise time equal to t_{lag} as indicated in figure 7.63 to the incident overpressure.

For the blast wave moving perpendicular to the long axis of the building the overpressure variation with time at all points is different as a result of vortex action. However, at any slab element the overpressure-time variation may be treated as constant across the span. If the response is rapid, i.e., before the vortex action takes place, the slab element is subjected to the incident overpressure uniformly distributed.

In this example the preliminary design loading for the roof slab is the incident overpressure. For checking the preliminary design both directions of loading are considered and two numerical integrations are presented (tables 7.24 and 7.25, page 167).

The design load as idealized from the computed loading shown by figure 7.57 is defined by:



$$B = 10 \text{ psi} = \frac{10(144)(18)}{1000} = 25.9 \text{ kips}$$

$$T = 0.38 \text{ sec}$$

$$H = \frac{BT}{2} = \frac{(25.9)0.38}{2} = 4.92 \text{ kip-sec} \quad \text{(par. 6-11)}$$

b. <u>Dynamic Design Factors.</u> (Refer to table 6.1.)

Elastic range:

$K_L = 0.53$, $K_M = 0.41$, $K_{LM} = 0.77$

$R_{1m} = 12M_{Ps}/L$, $k_1 = \dfrac{384EI}{L^3}$

$V = 0.36R + 0.14P$

Elasto-plastic range:

$K_L = 0.64$, $K_M = 0.50$, $K_{LM} = 0.78$

$R_m = \dfrac{8}{L}(M_{Ps} + M_{Pm})$, $k_{ep} = \dfrac{384EI}{5L^3}$

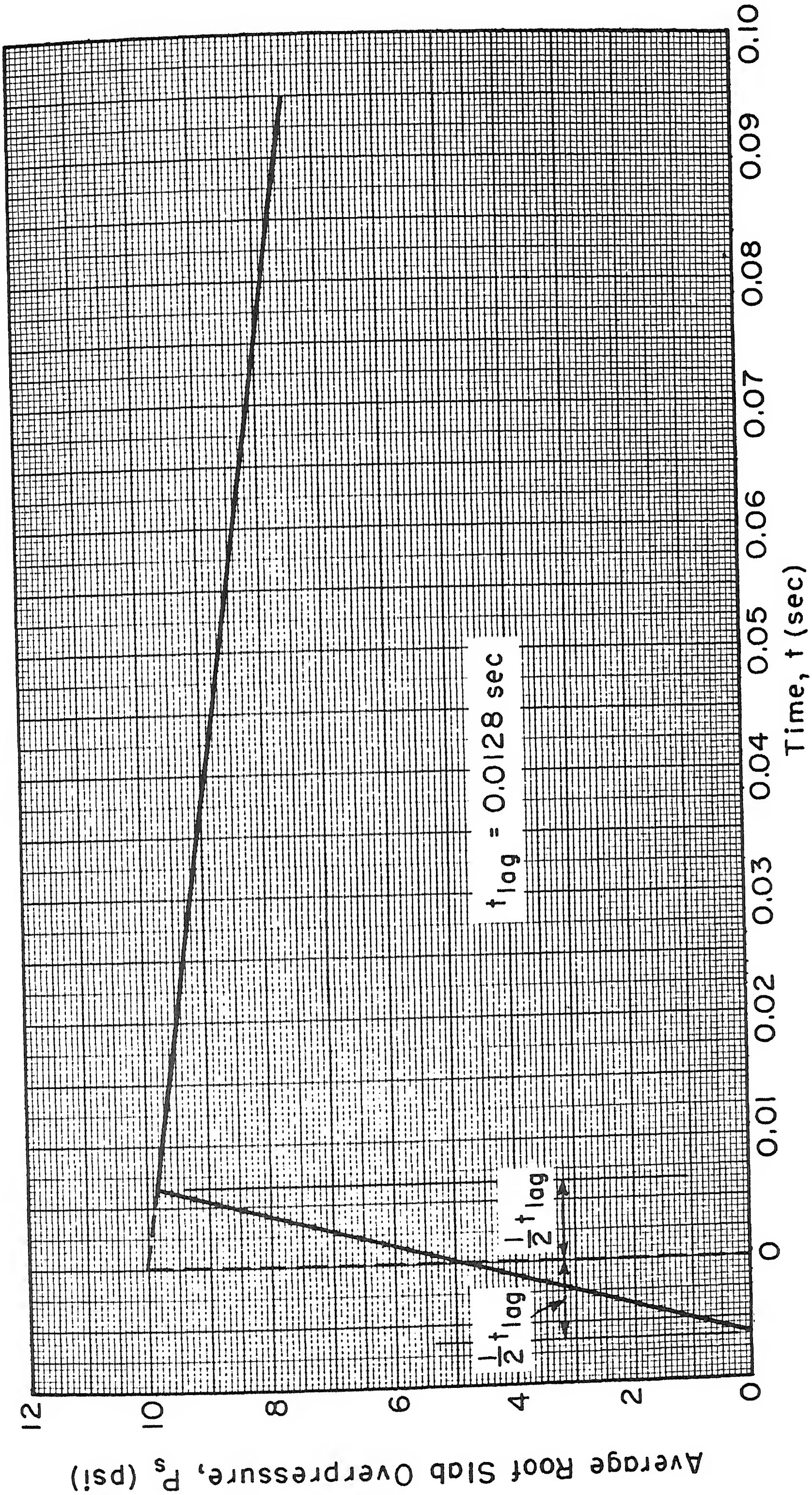


Figure 7.63. Average roof slab overpressure, blast wave parallel to slab span

Plastic range:

$K_L = 0.50,$ $\quad K_M = 0.33,$ $\quad K_{LM} = 0.66$

$$R_m = \frac{8}{L}(M_{Ps} + M_{Pm})$$

$$V = 0.38R + 0.12P$$

Average values:

$$K_L = 0.5(0.64 + 0.50) = 0.57$$

$$K_M = 0.5(0.50 + 0.33) = 0.42$$

$$K_{LM} = 0.5(0.78 + 0.77) = 0.77$$

$$R_m = \frac{8}{L}(M_{Ps} + M_{Pm})$$

$$k_E = 307EI/L^3$$

c. <u>First Trial - Actual Properties.</u>

Let $M_{Ps} = M_{Pm} = M_P$

Assume $p = 0.015$ (par. 4-10)

Let $\alpha\beta = 5$ (par. 6-26)

Assume $C_R = 1.0$ (experience)

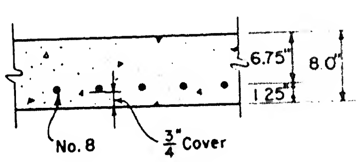
$R_m = C_R B = 1.0(25.9) = 25.9$ kips

$$M_P = pf_{dy}bd^2\left(1 - \frac{pf_{dy}}{1.7f'_{dc}}\right) \qquad \text{(eq 4.16)}$$

$$= 0.015(52)(1)d^2\left[1 - \frac{0.015(52)}{1.7(3.9)}\right] = 0.688d^2 \text{ kip-ft (d in inches)}$$

$$R_m = \frac{16M_P}{L} = \frac{(16)0.688d^2}{18} = 25.9, \quad \therefore\ d = 6.5 \text{ in.}$$

Try $h = 8$ in., $d = 6.75$ in., $p = 0.015$



$$M_P = 0.688(6.75)^2 = 31.4 \text{ kip-ft}$$

$$R_m = \frac{16M_P}{L} = \frac{16(31.4)}{18} = 27.9 \text{ kips}$$

$$I_g = bh^3/12 = (8.0)^3 = 511 \text{ in.}^4$$

$$I_t = bd^3\left[\frac{k^3}{3} + np(1-k)^2\right] = 12(d)^3\left[\frac{(0.42)^3}{3} + 0.15(1 - 0.42)^2\right]$$

$$= 0.905d^3 = 0.905(6.75)^3 = 278 \text{ in.}^4$$

$I_a = 0.5(I_g + I_t) = 0.5(511 + 278) = 394 \text{ in.}^4$

$k_E = \dfrac{307EI}{L^3} = \dfrac{(307)3(10)^3 394}{(18)^3 144} = 432 \text{ kips/ft}$

$y_E = \dfrac{R_m}{k_E} = \dfrac{27.9}{432} = 0.0645 \text{ ft}$

$y_m = \alpha\beta\, y_E = 5(0.0645) = 0.3225 \text{ ft (par. 6-26)}$

$\text{Weight} = \left[\dfrac{8(150)}{(12)} + 6\right] \dfrac{18}{1000} = 1.91 \text{-kips}$

$\text{Mass } m = \dfrac{1.91}{32.2} = 0.0591 \text{ kip-sec}^2\text{/ft}$

d. First Trial - Equivalent System Properties.

$R_{me} = K_L R_m = 0.57(27.9) = 15.9 \text{ kips (eq 6.12)}$

$H_e = K_L H = 0.57(4.92) = 2.8 \text{ kip-sec (eq 6.2)}$

$m_e = K_M m = 0.42(0.0591) = 0.0248 \text{ kip-sec}^2\text{/ft (eq 6.2)}$

$W_P = \dfrac{(H_e)^2}{2m_e} = \dfrac{(2.8)^2}{2(0.0248)} = 158 \text{ ft-kips (eq 6.10)}$

$T_n = 2\pi\sqrt{K_{LM} m/k_E} = 6.28\sqrt{0.77(0.0591)/432} = 0.0645 \text{ sec}$

e. Work Done vs Energy Absorption Capacity.

$C_T = T/T_n = 0.38/0.0645 = 5.9$

$C_R = R_m/B = 27.9/25.9 = 1.08 \text{ (eqs 6.15, 6.16)}$

$t_m/T = 0.17 \text{ (fig. 5.29)}$

$t_m = (0.17)0.38 = 0.0646 \text{ sec}$

Idealized load-time curve is satisfactory at $t = 0.0646$ sec (par. 5-13)

$C_W = 0.02 \text{ (fig. 5.27)}$

$W_m = C_W W_P = 0.02(158) = 3.16 \text{ ft-kips (eq 6.17)}$

$E = R_{me}(y_m - 0.5y_E) = 15.9\,[0.3225 - 0.5(0.0645)]$

$= 4.62 \text{ ft-kips (eq 6.18)}$

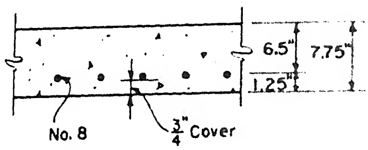
$E > W$, therefore the selected proportions are satisfactory as a preliminary design.

Try to reduce slab to bring E closer to W .

f. <u>Second Trial - Actual Properties.</u>

$$R_m = \frac{0.25W_m + 0.75E}{K_L(y_m - 0.5y_E)} = \frac{0.25(3.16) + 4.82(0.75)}{(0.57)[0.3225 - 0.5(0.0645)]} = 26.6 \text{ kips (eq 6.19)}$$

$$R_m = \frac{16M_P}{L} = \frac{(16)0.688d^2}{18} = 26.6, \therefore d = 6.6 \text{ in.}$$



Try h = 7.75 in., d = 6.50 in., p = 0.015

$$M_P = 0.688d^2 = 0.688(6.5)^2 = 29.1 \text{ kip-ft}$$

$$R_m = \frac{16M_P}{L} = \frac{16(29.1)}{18} = 25.9 \text{ kips}$$

$$I_g = \frac{bh^3}{12} = (7.75)^3 = 465 \text{ in.}^4$$

$$I_t = 0.905d^3 = 0.905(6.50)^3 = 248 \text{ in.}^4 \quad (k = 0.42)$$

$$I_a = 0.5(I_g + I_t) = 0.5(465 + 248) = 356 \text{ in.}^4$$

$$k_E = \frac{307EI}{L^3} = \frac{(307)3(10)^3 356}{(18)^3 144} = \text{[illegible] kips/ft}$$

$$y_E = \frac{R_m}{k_E} = \frac{25.9}{391} = 0.066 \text{ ft}$$

$$y_m = \alpha\beta\ y_E = 5(0.066) = \text{[illegible] ft (par. [illegible])}$$

$$\text{Weight} = \left[\frac{7.75(150)}{(12)} + \text{[illegible]}\right]\text{[illegible]}$$

$$\text{Mass } m = \frac{1.86}{32.2} = 0.0575 \text{ kip-[illegible]}$$

g. <u>Second Trial - Equivalent System Properties.</u>

$$R_{me} = K_L R_m = 0.57(25.9) = \text{[illegible] kips ([illegible])}$$
$$H_e = K_L H = 0.57(4.92) = \text{[illegible] kip-[illegible] ([illegible])}$$
$$m_e = K_M m = 0.42(0.0575) = \text{[illegible]} \quad \text{([illegible])}$$

$$W_P = \frac{(H_e)^2}{2m_e} = \frac{(2.8)^2}{2(0.0242)} = \text{[illegible] ft-kips ([illegible])}$$

$$T_n = 2\pi\sqrt{K_{LM}m/k_E} = 6.28\sqrt{0.77(0.0575)/391} = 0.0667 \text{ sec}$$

h. Work Done vs Energy Absorption Capacity.

$C_T = T/T_n = 0.38/0.0667 = 5.7$

$C_R = R_m/B = 25.9/25.9 = 1.0$ (eqs 6.15, 6.16)

$t_m/T = 0.24$ (fig. 5.29)

$t_m = (0.24)0.38 = 0.091$ sec

Idealized load-time curve is satisfactory at time t = 0.091

par. 5-13)

$C_W = 0.028$ (fig. 5.27)

$W_m = C_W W_p = 0.028(162) = 4.55$ ft-kips (eq 6.17)

$E = R_{me}(y_m - 0.5y_E) = 14.75\ [0.330 - 0.5(0.066)]$

$= 4.38$ ft-kips (eq 6.18)

$E \approx W$, therefore the selected proportions are satisfactory as a pre-minary design.

i. Preliminary Design for Bond Stress.

Estimated $V_{max} = 0.5R_m = 0.5(25.9) = 12.9$ kips

Allowable $u = 0.15f'_c = 0.15(3000) = 450$ psi (par. 4-09)

$$\Sigma o = \frac{V}{ujd} = \frac{8(12,900)}{450(7)6.5} = 5.04 \text{ in.}$$

Try #7 at 6 in., $A_s = 1.2$ in.^{2}, $\Sigma o = 5.5$ in., $p = \frac{A_s}{bd} = \frac{1.2}{12(6.56)} = 0.0152$

$np = 10(0.0152) = 0.152$, $h = 7.75$ in., $d = 6.56$ in.

j. Determination of Maximum Deflection and Dynamic Reactions by merical Integration.

$$M_P = pf_{dy}bd^2\left[1 - \frac{pf_{dy}}{1.7f'_{dc}}\right] = 0.0152(52)(1)(6.56)^2\left[1 - \frac{(0.0152)52}{1.7(3.9)}\right]$$

$= 30.4$ kip-ft (eq 4.16)

$I_g = bh^3/12 = (7.75)^3 = 465$ in.^{4}

$I_t = bd^3\left[\frac{K^3}{3} + np(1 - k)^2\right] = 0.905(d^3) = 0.0905(6.56)^3 = 256$ in.^{4}

$I_a = 0.5(I_g + I_t) = 0.5(465 + 256) = 360$ in.^{4}

$$\text{Weight} = \left[\frac{7.75(150)}{12} + 6\right]\frac{18}{1000} = 1.86 \text{ kips}$$

$$\text{Mass } m = \frac{1.86}{32.2} = 0.0575 \text{ kip-sec}^2/\text{ft}$$

Elastic range:

$$R_{lm} = \frac{12M_P}{L} - \text{weight} = \frac{12(30.4)}{18} - 1.86 = 18.3 \text{ kips}$$

$$k_1 = \frac{384EI}{L^3} = \frac{(384)3(10)^3 360}{18^3(144)} = 480 \text{ kips/ft}$$

$$y_e = \frac{R_{lm}}{k_1} = \frac{18.3}{480} = 0.0381 \text{ ft}$$

Elasto-plastic range:

$$R_m = \frac{16M_P}{L} - \text{weight} = \frac{16(30.4)}{18} - 1.86 = 25.1 \text{ kips}$$

$$k_{ep} = \frac{384EI}{5L^3} = \frac{1}{5}k_1 = 96 \text{ kips/ft}$$

$$y_{ep} = y_e + \frac{R_m - R_{lm}}{k_{ep}} = 0.0381 + \frac{25.1 - 18.3}{96} = 0.0381 + 0.071 = 0.109 \text{ ft}$$

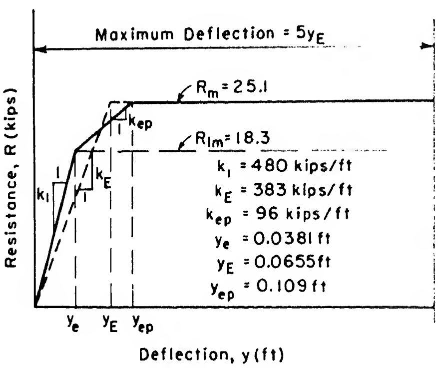


Figure 7.64. Resistance function for 7-3/4-in. slab spanning 18 ft, fixed at both ends

Plastic range:

$$R_m = \frac{16M_P}{L} - \text{weight} = 25.1 \text{ kips}$$

$$k_E = \frac{307EI}{L^3} = \frac{307}{384}k_1 = 383 \text{ kips/ft}$$

$$y_E = \frac{R_m}{k_E} = \frac{25.1}{383} = 0.0655 \text{ ft}$$

$$y_m = \alpha\beta\, y_e = 5(0.0655) = 0.3275 \text{ ft}$$

$$T_n = 2\pi\sqrt{K_{LM}m/k_E}$$

$$= 6.28\sqrt{0.77(0.0575)/383}$$

$$= 0.0674 \text{ sec}$$

The basic equation for the numerical integrations in tables 7.24 and 7.25 is $y_{n+1} = \ddot{y}_n(\Delta t)^2 + 2y_n - y_{n-1}$ (table 5.3) where

Table 7.24. Determination of Maximum Deflection and Dynamic Reactions for Roof Slab, Incident Overpressure (Modified for Rise Time)

t (sec)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)	V_n (kips)
0	0	0	2.0	0.0022	0	0
0.007	13.0	1.0	12.0	0.0132	0.0022	2.2
0.014	24.5	8.4	16.1	0.0178	0.0176	6.5
0.021	24.0	19.5	4.5	0.0049	0.0508	10.3
0.028	23.6	23.2	0.4	0.0004	0.0889	11.6
0.035	23.1	25.1	-2.0	-0.0026	0.1274	12.3
0.042	22.6	25.1	-2.5	-0.0032	0.1633	12.3
0.049	22.2	25.1	-2.9	-0.0037	0.1960	12.2
0.056	21.7	25.1	-3.4	-0.0044	0.2250	12.1
0.063	21.3	25.1	-3.8	-0.0049	0.2496	12.1
0.070	20.8	25.1	-4.3	-0.0056	0.2693	12.0
0.077	20.4	25.1	-4.7	-0.0061	0.2834	12.0
0.084	19.9	25.1	-5.2	-0.0067	0.2914	11.9
0.091	19.5	25.1	-5.6	-0.0072	0.2927*	11.9
					0.2868	

$*(y_n)_{max} = 0.29.$

Table 7.25. Determination of Maximum Deflection and Dynamic Reactions for Roof Slab, Incident Overpressure (No Rise Time)

t (sec)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)	V_n (kips)
0	25.9	0	12.95	0.01373	0	3.6
0.00685	25.4	6.6	18.8	0.01993	0.01373	4.9
0.01370	24.9	19.2	5.7	0.00596	0.04739	10.2
0.02055	24.4	23.0	1.4	0.00146	0.08701	11.6
0.02740	24.0	25.1	-1.1	-0.00136	0.12809	12.4
0.03425	23.4	25.1	-1.7	-0.00211	0.16781	12.4
0.04110	22.8		-2.3	-0.00285	0.20542	12.3
0.04795	22.4		-2.7	-0.00335	0.24018	12.2
0.05480	22.0		-3.1	-0.00384	0.27159	12.2
0.06165	21.6		-3.5	-0.00434	0.29916	12.1
0.06800	21.1		-4.0	-0.00496	0.32239	12.1
0.07535	20.7		-4.4	-0.00545	0.34066	12.0
0.08220	20.7		-4.4	-0.00545	0.35348	12.0
0.08905	20.2		-4.9	-0.00607	0.36085	12.0
0.09590	19.9		-5.2	-0.00644	0.36215*	11.9
0.10275	18.9	22.6			0.35701	10.8

$*(y_n)_{max} = 0.362.$

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(\Delta t)^2}{K_{LM}(m)} = \frac{(P_n - R_n)(0.007)^2}{K_{LM}(m)}$$

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(49)10^{-6}}{0.77(0.0575)} = 1.107(10^{-3})(P_n - R_n) \text{ ft, elastic range}$$

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(49)10^{-6}}{0.78(0.0575)} = 1.093(10^{-3})(P_n - R_n) \text{ ft, elasto-plastic range}$$

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(49)10^{-6}}{0.66(0.0575)} = 1.291(10^{-3})(P_n - R_n) \text{ ft, plastic range}$$

The time interval $\Delta t = 0.007$ sec is approximately $T_n/10 = 0.00674$ sec (par. 5-08). The dynamic reaction equations are listed in paragraph 7-41b. In table 7.24 the slab analysis considers the loading resulting from the shock wave moving along the long axis of the building (incident overpressure modified for rise time). The P_n values for the second column are obtained from figures 7.57 and 7.63, multiplying by 144(18)/1000 = 2.59. The slab is not critical for this condition $(y_n)_{max} = 0.2927 < 0.3275$.

In table 7.25 the slab analysis considers the loading resulting from the shock wave moving along the short axis of the building (local roof overpressure with no time of rise). The P_n values for the second column are obtained from figure 7.57, multiplying by 2.59. The slab is critical for this case and in fact the maximum deflection $(y_n)_{max} = 0.3621 > 0.3275$. This is accepted as a satisfactory design since

$$\alpha\beta = \frac{0.3621}{0.0655} = 5.5$$

is close enough to value of 5 established above.

k. <u>Shear Strength and Bond Stress.</u>

$V_{max} = 12.4$ kips (table 7.25)

For no shear reinforcement

Allowable $v_y = 0.04f'_c + 5000p$ (eq 4.34)

$v_y = 0.04(3000) + 5000(0.0152) = 120 + 76 = 196$ psi

$$v = \frac{8V}{7bd} = \frac{8(12,400)}{7(12)(6.56)} = 178 \text{ psi; OK, no shear reinforcement required}$$

$$u = \frac{8V}{7\Sigma od} = \frac{8(12,400)}{7(5.5)6.56} = 390 \text{ psi}$$

Allowable u = $0.15f'_c = 0.15(3000) = 450$ psi; OK

1. Summary.

7-3/4-in. slab

$p = 0.0152$

$\Sigma o = 5.5$ in.

No shear reinforcement

7-42 PRELIMINARY COLUMN DESIGN. It has been found desirable to make a preliminary design of the column before designing the girders because the column resisting moment is a factor in design of the girder (par. 7-11).

A single-story frame subject to lateral load behaves essentially as a single-degree-of-freedom system. In determining the requirements for the columns which are the springs of this system it is not necessary to use the equivalent system technique used in designing all elements.

Using the principles of paragraph 6-11 and equations from paragraph -06 provides a procedure for obtaining preliminary column sizes.

In the preliminary design the girders are assumed to be infinitely rigid to simplify the analysis. In determining the spring constant, the column height is 14.75 ft from the centerline of the girder to the top of the footing. The resistance computation is based on the clear height $_c$ = 13.0 ft.

If the first trial section is overstrength or understrength it is desirable to make a second trial. In this plastic column design, since the adequacy of the selected section is based on a comparison of E , the energy absorption capacity, with W_m, the work done on the frame, successive trials are obtained by estimating a design energy level somewhere between the computed values of E and W for use in the equation

$$R_m = \frac{\text{energy}}{x_m}$$

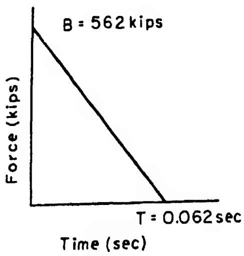
or determining the new trial R . If $W_m > E$ the energy level should be set at W_m so that

$$R_m = \frac{W_m}{x_m}$$

However, if $E > W_m$, as in the following example, the intermediate energy level may be obtained as illustrated on the following page.

a. <u>Design Loading.</u> The design lateral load on the frame is obtained from the dynamic reactions at the top of the front wall slab. However, for a preliminary design it is satisfactory to use the net lateral overpressure curve (fig. 7.60).

In computing the total concentrated lateral load on the frame it is assumed that the wall slab transmits the blast loads equally to the roof slab and foundation. This is generally conservative for the frame because, in the elastic phase, the dynamic reaction at the roof is less than that at the foundation (table 6.1).



The design load as idealized from the computed loading shown by figure 7.60 is defined by:

$$B = 25.3 \text{ psi} = \frac{25.3(144)18\left[\frac{15.83}{2} + 0.65\right]}{1000}$$

$$= 562 \text{ kips}$$

$$T = 0.062 \text{ sec}$$

$$H = \frac{BT}{2} = \frac{(562)0.062}{2} = 17.4 \text{ kip-sec (par. 6-11)}$$

b. <u>Mass Computation.</u>

Roof slab $\left[\frac{7.75(150)}{12} + 6.0\right]\frac{18(44.25)}{1000} = 82.0$ kips

Girder (assumed) $\frac{18(32)42(150)}{12(12)1000} = 25.2$ kips

3 columns (assumed) $\frac{12(24)13(150)3}{12(12)1000} = 11.7$ kips

2 wall slabs $\frac{10.5(18)15.83(150)2}{12(1000)} = 74.8$ kips

Mass of single-degree-of-freedom system = total (roof + girder) +

$$1/3(\text{columns} + \text{wall}) = \frac{82.0 + 25.2 + 0.33(11.7 + 74.8)}{32.2} = 4.22 \text{ kip-sec}^2/\text{ft}$$

c. <u>First Trial - Actual Properties.</u>

Assume $p = 0.015$ (par. 4-10)

Let $\alpha\beta = 6$ (par. 6-26)

Assume $C_R = 0.50$ (experience)

$R_m = C_R B = 0.50(562) = 281$ kips

$$\text{Required } M_D = \frac{R_m h}{2n} = \frac{281(13.0)}{2(3)} = 610 \text{ kip-ft}$$

Approximate average roof pressure = 6.5 psi (estimated in fig. 7.61)

Average blast load per column $= \dfrac{44.25(18)6.5(144)}{3(1000)} = 248$ kips

Dead load per column $= 1/3(82.0 + 25.2) = 36.0$ kips

Average column design load $P_D = 248 + 36 = 284$ kips

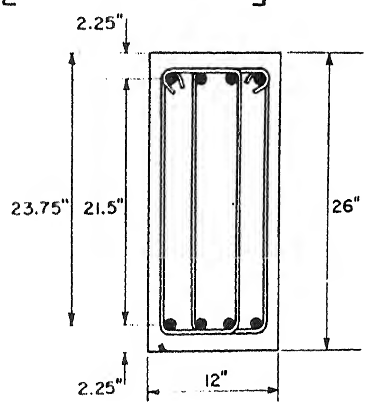
$$M_D = A_s f_{dy} d' + P_D\left(0.5t - \frac{P_D}{1.7 f'_{dc}}\right) \quad \text{(eq 4.32)}$$

Let $p = p' = 0.015$, $d' = (t - 4.5)$ in., $d'' = 2.25$ in.

$b = 12$ in., $A_s = pbt$

Substituting into previous M_D equation

$$610(12) = 0.015(12)t(52)(t - 4.5) + 284\left[0.5t - \frac{284}{1.7(12)3.9}\right]$$



$$9.35t^2 - 42t + 142t - 1015 - 7320 = 0$$

Solving $t = 25.0$ in.

Try $b = 12$ in., $t = 26$ in.

$$M_D = \frac{0.015(12)26(52)(26 - 4.5)}{12} + \frac{284}{12}\left[0.5(26) - \frac{284}{1.7(12)3.9}\right]$$

$M_D = 436 + 223 = 659$ kip-ft

$d''/d = 2.25/23.75 = 0.0948$

For $p = p' = 0.015$ and $n = 10$

$m = np + (n - 1)p' = 0.15 + 9(0.015) = 0.285$

$$q = np + (n - 1)p' \frac{d''}{d} = 0.15 + 9(0.015)0.0948 = 0.1628$$

$k = 0.35$ (table 11 RCDH of ACI)

$$I_t = bd^3\left[\frac{k^3}{3} + (n - 1)p'\left\{k^2 - 2k\left(\frac{d''}{d}\right) + \left(\frac{d''}{d}\right)^2\right\} + np(1 - k)^2\right]$$

$$= 12(23.75)^3\left[\frac{(0.35)^3}{3} + 9(0.015)\left\{(0.35)^2 - 2(0.35)(0.0948) + (0.0948)^2\right\} + 0.15(1 - 0.35)^2\right] = 13{,}800 \text{ in.}^4$$

$I_g = bt^3/12 = 26^3 = 17{,}576$ in.^{4}

$I_a = 0.5(I_g + I_t) = 15{,}700$ in.^{4}

$$k = \frac{12EIn}{h^3} = \frac{12(3)10^3(15{,}700)3}{(14.75)^3 144} = 3670 \text{ kips/ft}$$

$$R_m = \frac{2n}{h_c} M_D = \frac{2(3)659}{13.0} = 304 \text{ kips}$$

$$x_e = R_m/k = 304/3670 = 0.083 \text{ ft}$$

$$x_m = \alpha\beta\, x_e = 6x_e = 6(0.083) = 0.498 \text{ ft (par. 6-26)}$$

$$T_n = 2\pi\sqrt{m/k} = 6.28\sqrt{4.22/3670} = 0.213 \text{ sec}$$

d. First Trial - Work Done vs Energy Absorption Capacity.

$T/T_n = 0.062/0.213 = 0.291$

$C_R = R_m/B = 304/562 = 0.54$

$t_m/T = 1.35$ (fig. 5.29), $t_m = 1.35(0.062) = 0.084$ sec

The original load-time curve should be revised to obtain a closer approximation to the total impulse up to time t_m. The impulse up to $t = 0.10$ sec in figure 7.60 is $H = 1.167$ psi-sec (obtained by graphical integration).

$T = 2H/B = 2(1.167)/25.3 = 0.0923$ sec

$T/T_n = 0.0923/0.213 = 0.437$

$t_m/T = 1.2$ (fig. 5.29), $t_m = 1.2(0.0923) = 0.111$ sec

Try again for impulse up to $t = 0.12$ sec

$H = 1.214$ psi-sec, $T = 2(1.214)/25.3 = 0.096$ sec, $T/T_n = 0.096/0.213 = 0.45$

$t_m/T = 1.2$ (fig. 5.29), $t_m = 1.2(0.096) = 0.115$ sec; OK

$C_W = 0.71$ (fig. 5.27)

$$W_P = \frac{H^2}{2m} = \frac{(BT/2)^2}{2m} = \frac{(562)^2(0.096)^2}{8(4.22)} = 86.5 \text{ ft-kips}$$

$$W_m = C_W W_P = 0.71(86.5) = 61.5 \text{ ft-kips}$$

$$E = R_m(x_m - 0.5x_e) = 304\,[0.498 - 0.5(0.083)] = 139 \text{ ft-kips}$$

Since $E \gg W_m$ a new trial should be made.

e. Second Trial - Actual Properties. Since $E \gg W_m$, the intermediate value of energy for use in determining the second trial R_m is obtained as follows:

$$R_m = \frac{0.5(W_m + E)}{x_m} = \frac{0.5(61.5 + 139.0)}{0.498} = 201 \text{ kips}$$

$$\text{Required } M_D = \frac{R_m h_c}{2n} = \frac{201(13.0)}{2(3)} = 436 \text{ kip-ft}$$

Use same column constants as above and substitute into equation (4-32)

$$436(12) = 0.015(12)t(52)(t - 4.5) + 284\left[0.5t - \frac{284}{1.7(12)3.9}\right]$$

$$9.35t^2 - 42t + 142t - 1015 - 5240 = 0$$

$t = 21$ in., try $b = 12$ in., $t = 20$ in.

$$M_D = \frac{0.015(12)20(52)(20 - 4.5)}{12} + \frac{284}{12}\left[10 - \frac{284}{1.7(12)3.9}\right]$$

$$= 242 + 152 = 394 \text{ kip-ft}$$

$d''/d = 2.25/(20 - 2.25) = 0.127$

For $p = p' = 0.015$ and $n = 10$

$m = np + (n - 1)p' = 0.15 + 9(0.015) = 0.285$

$q = np + (n - 1)p\, d''/d = 0.15 + 9(0.015)0.127 = 0.167$

$k = 0.36$ (table 11 RCDH of ACI)

$$I_t = bd^3\left[\frac{k^3}{3} + (n - 1)p'\left\{k^2 - 2k\left(\frac{d''}{d}\right) + \left(\frac{d''}{d}\right)^2\right\} + np(1 - k)^2\right]$$

$$= 12(17.75)^3\left[\frac{(0.36)^3}{3} + 9(0.015)\left\{(0.36)^2 - 2(0.36)(0.127) + (0.127)^2\right\} + 0.15(1 - 0.36)^2\right]$$

$I_t = 67{,}200(0.0845) = 5680 \text{ in.}^4$
$I_g = bt^3/12 = (20^3) = 8000 \text{ in.}^4$
$I_a = 0.5(I_g + I_t) = 6840 \text{ in.}^4$

$$k = \frac{12EIn}{h^3} = \frac{12(3)10^3(6840)3}{(14.75)^3 144} = 1590 \text{ kips/ft}$$

$$R_m = \frac{2nM_D}{h_c} = \frac{2(3)394}{13.0} = 182 \text{ kips}$$

$x_e = R_m/k = 182/1590 = 0.114$ ft

$x_m = \alpha\beta\, x_e = 6x_e = 6(0.114) = 0.684$ ft (par. 6-26)

$T_n = 2\pi\sqrt{m/k} = 6.28\sqrt{4.22/1590} = 0.322$ sec

f. Second Trial - Work Done vs Energy Absorption Capacity.

$T/T_n = 0.096/0.322 = 0.298$

$C_R = R_m/B = 182/562 = 0.324$

$t_m/T = 1.8$ (fig. 5.29), $t_m = 1.8(0.096) = 0.173$ sec

The value of t_m exceeds the value of t = 0.12 sec for which the value of T = 0.096 was obtained. The idealized load-time curve should be revised to approximate more closely the total impulse up to t_m. The total impulse up to t = 0.20 sec (fig. 7.60) is H = 1.374 psi-sec (obtained by graphical integration).

$$T = 2H/B = 2(1.374)/25.3 = 0.109 \text{ sec}$$

$$T/T_n = 0.109/0.322 = 0.338$$

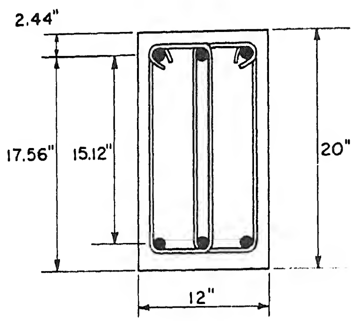
$$t_m/T = 1.75 \text{ (fig. 5.29)}, \quad t_m = 1.75(0.109) = 0.191 \text{ sec; OK}$$

$$C_W = 0.83 \text{ (fig. 5.27)}$$

$$W_P = \frac{H^2}{2m} = \frac{(BT/2)^2}{2m} = \frac{(562)^2(0.109)^2}{8(4.22)} = 111.0 \text{ ft-kips}$$

$$W_m = C_W W_P = 0.83(111.0) = 92.1 \text{ ft-kips}$$

$$E = R_m(x_m - 0.5x_e) = 182\,[0.684 - 0.5(0.114)] = 114 \text{ ft-kips}$$



E is slightly greater than W and another trial might be made. It is desirable, however, to be conservative in the preliminary design because simplifying assumptions are used (par. 7-44). Therefore the 12 by 20 column is selected as the preliminary design and an actual column section is selected to establish the column plastic bending moment for use in the girder design that follows in paragraph 7-43.

d'' = 2.44 in., d = 17.56 in., d' = 15.12 in., 3 #9 bars,

$$A_s = A_s' = 3 \text{ in.}^2$$

$$M_D = A_s f_{dy} d' + P_D \left[0.5t - \frac{P_D}{1.7(b)f'_{dc}} \right]$$

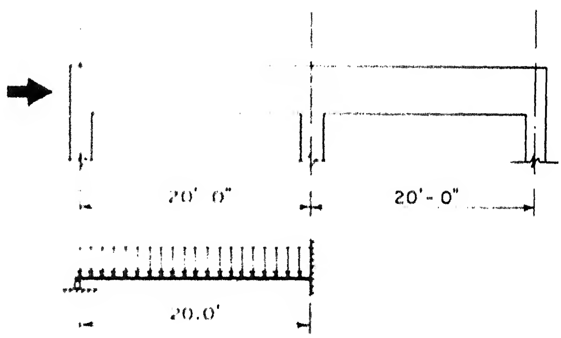
$$= \frac{3(52)15.12}{12} + \frac{284}{12}\left[10 - \frac{284}{1.7(12)3.9}\right] = 197 + 237 - 84.5 = 350 \text{ kip-ft}$$

7-43 DESIGN OF ROOF GIRDER. The frame of this building consists of three rectangular columns supporting a rectangular girder which forms a tee beam with the roof slab. The roof girder is designed to resist the combined vertical loads on the roof and the lateral loads on the frame as explained

paragraph 7-11. Since this manual provides a technique for handling single-span elements, the continuous girder is designed to withstand ical loads as a beam fixed at the interior support and pinned at the rior support.

Although the other structural elements of this building are permitted eflect plastically, the girder should be designed to behave elastically hat proper restraint is maintained for the column throughout the detion history of the frame.

The design bending moment of girder is the sum of the moments he fixed support due to (1) the slab dynamic reactions, (2) the ic loads, and (3) the frame ac-. The moment due to frame ac- in a two-bay frame is equal to half the column plastic moment 7-11).



The girder is designed as a tee beam in the region of positive mo- In the region of negative moment near the interior support the girder rectangular section. This results in a girder of variable moment of ia for which there are special expressions for R_m, y_m, and k_1 (par. and table 6.4).

a. Loading. The critical frame girder loading results from the wave traveling parallel to the girder. Since the girder is parallel e short side of the building the appropriate roof loading is from the 3 condition. For Zone 3 loading the time variation of the local roof ressure varies continuously from front to back of the building.

To obtain the true variation of slab dynamic reaction on the girder be a tedious task as well as unjustified in the light of all the pos- inaccuracies. In this example a conservative loading based on the ent overpressure curve (fig. 7.57) is used as the basis for the slab in determining the girder load. This is warranted because in general lab reaches its maximum displacement before the vortex action in 3 has an opportunity to reduce the incident overpressure to the local

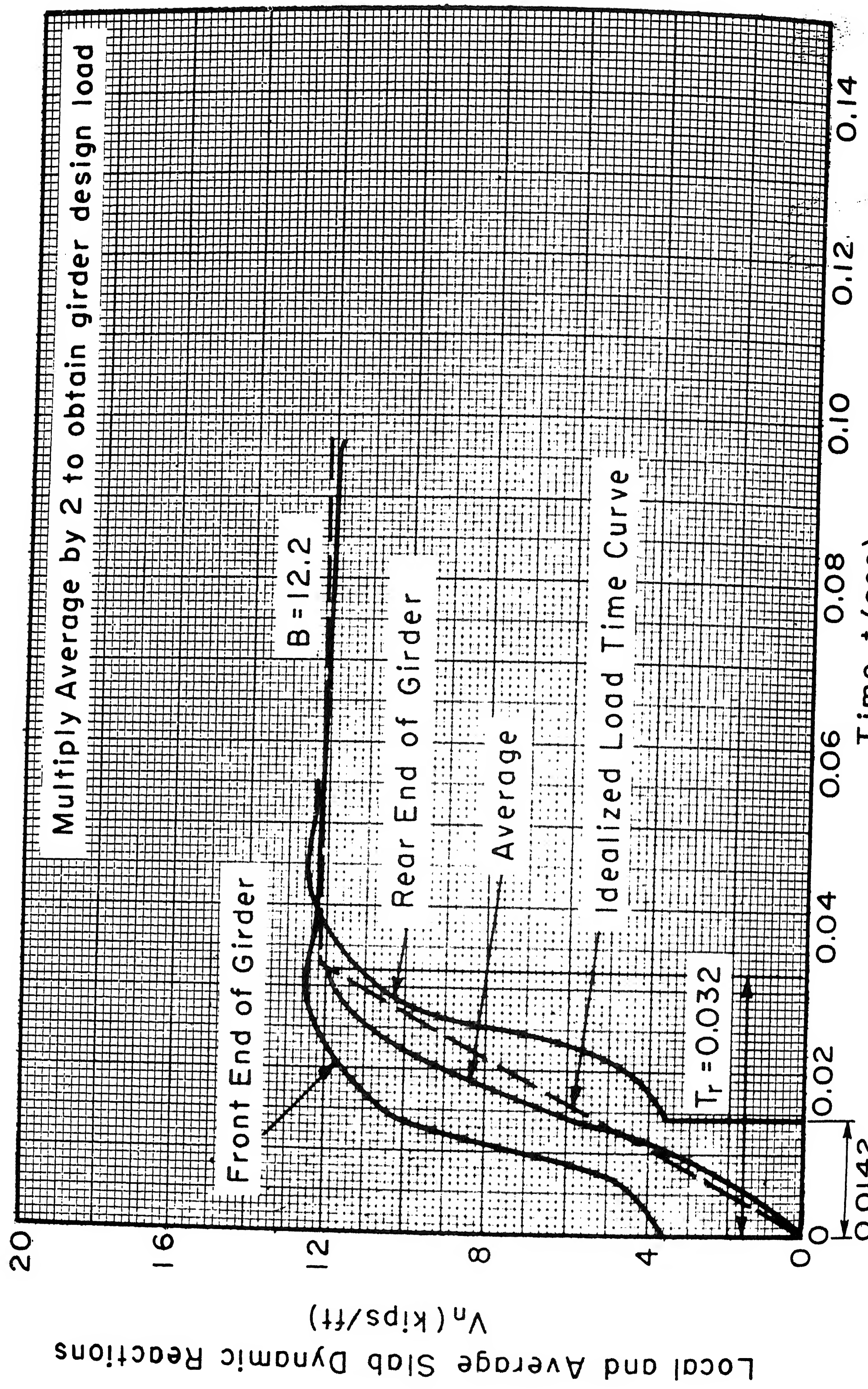
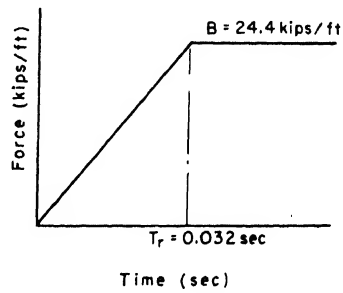


Figure 7.65. Local and average slab dynamic reactions for incident overpressure, blast wave parallel ...

roof overpressure (see pars. 7-23 and 7-25 and table 7.5).

The girder load is determined from the slab dynamic reactions for incident overpressure with the blast wave traveling parallel to the girder. Plotting from table 7.25 the same variation of dynamic reaction at the front and rear ends of the girder and averaging the total load on the girder gives the variation of average girder load from one slab in figure 7.65. The preliminary design load is obtained by idealizing the load-time curve in figure 7.65 and multiplying by two to account for the two slabs loading the girder. The idealized load is



$B = 12.2(2) = 24.4$ kips/ft

$T_r = 0.032$ sec

b. Elastic Range Dynamic Design Factors. (Refer to table 6.4.)

$K_L = 0.58,$ $\quad K_M = 0.45,$ $\quad K_{LM} = 0.78$

$k = f_3 EI_1/L^3$ (fig. 6.29)

$R_m = f_1 M_{Ps}/L$ (fig. 6.27)

$M_{pos} = f_2 M_n$ (fig. 6.28)

$M_n = R_1 L/f_1$ (fig. 6.27)

$V_1 = 0.26R + 0.12P$

$V_2 = 0.43R + 0.19P$

c. Mass Computation.

$$\text{Slab and roofing} = \left[\frac{7.75(150)}{12} + 6.0\right]\frac{18(44.25)}{1000(2)} = 41.0 \text{ kips}$$

$$\text{Girder (estimate)} = \frac{18(36)20(150)}{(12)12(1000)} = 13.5 \text{ kips}$$

$$\text{Total mass} = \frac{41.0 + 13.5}{32.2} = 1.69 \text{ kip-sec}^2/\text{ft}$$

d. First Trial - Actual Properties.

Estimate tee beam action for first trial,

$f_1 = 9,$ $\quad f_2 = 0.67,$ $\quad f_3 = 240$

Assume D.L.F. = 1.5 (experience)

$R_m = \text{D.L.F.}(B) = 1.5(24.4)(20) = 732$ kips

Moment in girder at interior support (fixed support) due to static loads

$M = WL/8 = (41.0 + 13.5)20/8 = 136$ kip-ft

Moment in girder at midspan due to static load

$M = 9WL/128 = 9(54.5)20/128 = 77$ kip-ft

Moment from column

$M = 0.5M_p = 0.5(350) = 175$ kip-ft (par. 7-42f)

Moment resistance required for vertical blast loads

$M = R_m L/9 = 732(20)/9 = 1630$ kip-ft

The total moment resistance required

$1630 + 175 + 136 = 1941$ kip-ft

$$M_p = pf_{dy}bd^2\left(1 - \frac{pf_{dy}}{1.7f'_{dc}}\right) \quad \text{(eq 4.16)}$$

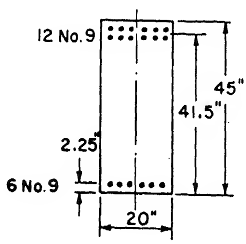
Assume $p = 0.015$

$$M_p = 0.015(52)bd^2\left[1 - \frac{0.015(52)}{1.7(3.9)}\right] = 0.688bd^2 = 1941(12)$$

$$= 23{,}300 \text{ kip-in.}$$

If $b = 20$ in.,, $d = 41.2$ in.; try $h = 45$ in., $d = 41.5$ in.

From figure 6.28 estimate $f_2 = 0.67$ (from experience)

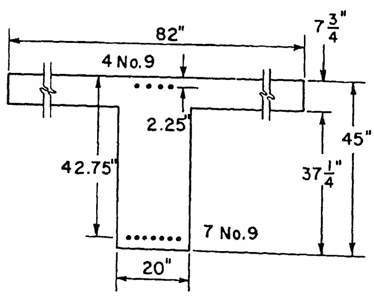


Effective Section at Interior Support

Midspan moment $= 0.67(1630) + 0 + 77$

$= 1090 + 77 = 1167$ kip-ft

Ratio of midspan tension reinforcement to interior support tension reinforcement is $\frac{1167}{1941} = 0.60.$



Effective Section at Midspan

Rectangular section at fixed support

$np = 10(12)/20(41.5) = 0.1445$

$np' = 10(6)/20(41.5) = 0.0722$

$I_g = 20(45)^3/12 = 152{,}000 \text{ in.}^4$

$m = np + (n - 1)p'$

$= 0.1445 + \frac{9}{10}(0.0722) = 0.2095$

$q = np + (n - 1)p'\frac{d'}{d}$

$= 0.1445 + \frac{9}{10}(0.0722)\frac{2.25}{41.5} = 0.148$

$k = 0.38$ (table 11 RCDH of ACI)

$kd = 0.38(41.5) = 15.7$

$I_t = 20(15.7)^3/3 + 9(6)(13.45)^2 + 12.0(10)(25.8)^2$

$= 19,350 + 9750 + 80,000 = 109,000 \text{ in.}^4$

$I_1 = (I_g + I_t)0.5 = 130,000 \text{ in.}^4$

Tee section at midspan

$np = 10(7)/82(42.75) = 0.0200$

$np' = 10(4)/82(42.75) = 0.0114$

$$\bar{y} = \frac{[-82(7.75)^2]/2 + [20(37.25)^2]/2}{82(7.75) + 20(37.25)} = \frac{11,640}{1380} = 8.45 \text{ in.}$$

$$I_g = \frac{82(7.75)^3}{3} + \frac{20(37.25)^3}{3} - 1380(8.45)^2$$

$= 12,700 + 345,000 - 98,500 = 250,200 \text{ in.}^4$

$m = np + (n - 1)p' = 0.0200 + 0.9(0.0114) = 0.0303$

$$q = np + (n - 1)p' \frac{d'}{d} = 0.0200 + 0.9(0.0114) \frac{2.25}{42.75} = 0.0205$$

$k = 0.18$ (table 11 RCDH of ACI), $kd = (0.18)(42.75) = 7.7 \text{ in.}$

$$I_t = \frac{82(7.7)^3}{3} + 9(4)(7.7 - 2.25)^2 + 10(7)(42.75 - 7.7)^2$$

$= 12,500 + 1070 + 86,000 = 99,570 \text{ in.}^4$

$I_2 = 0.5(I_g + I_t) = 179,400 \text{ in.}^4$

$I_1/I_2 = 130,000/179,400 = 0.725$

$f_1 = 8.85$ (fig. 6.27)

$f_2 = 0.66$ (fig. 6.28)

$f_3 = 235$ (fig. 6.29)

$$k_1 = \frac{f_3 EI_1}{L^3} = \frac{235(3)10^3(130,000)}{(20)^3 144} = 79,500 \text{ kips/ft}$$

At the interior support

$$M_P = pf_{dy}bd^2\left(1 - \frac{pf_{dy}}{1.7f'_{dc}}\right) \quad \text{(eq 4.16)}$$

$$= \frac{0.01445(52)20(41.5)^2}{12}\left[1 - \frac{0.01445(52)}{1.7(3.9)}\right] = 1910 \text{ kip-ft}$$

$$T_n = 2\pi\sqrt{\frac{K_{LM}m}{k_1}} = 6.28\sqrt{\frac{0.78(1.69)}{79,500}} = 0.0256 \text{ sec}$$

$$T_r/T_n = 0.032/0.0256 = 1.25$$

$$\left.\begin{array}{l} \text{D.L.F.} = 1.18 \\ t_m/T_r = 1.3 \end{array}\right\} \text{(fig. 5.21)}$$

$$t_m = 1.3(0.032) = 0.0417 \text{ sec}$$

The idealized loading is a satisfactory approximation for the loading in figure 7.65.

Required R = 1.18(24.4)20 = 575 kips

The available resisting moment at the interior support for vertical blast loads is

$$M = 1910 - 136 - 197 = 1577 \text{ kip-ft}$$

The available resistance is

$$R_{1m} = f_1 M_{Ps}/L = 8.9(1577)/20 = 700 \text{ kip-ft}$$

The available R_{1m} is greater than the required R_{1m}. Another trial will be made by reducing the amount of reinforcing steel. Reducing the steel required by the ratio of the resistance calculated above

Use 10 #9 tension bars; $A_s = (575/700)(12) = 9.85 \text{ in.}^2$

$$p = (10)/20(41.5) = 0.01205$$

$$M_P = \frac{0.01205(52)20(41.5)^2}{12}\left[1 - \frac{0.01205(52)}{1.7(3.9)}\right] = 1635 \text{ kip-ft}$$

Assuming no substantial change in I_1/I_2 and T_n the available resistance is $R_{1m} = 8.9(1635 - 197 - 136)/20 = 580 \text{ kips} > 575 \text{ kips}$

e. <u>Second Trial - Actual Properties.</u>

<u>Rectangular section at fixed support</u>

$$np = 10(10)/20(41.5) = 0.1205$$

$$np' = 10(6)/20(41.5) = 0.0702$$

$$I_g = 152,000 \text{ in.}^4$$

$$m = np + (n - 1)p' = 0.1205 + \frac{9}{10}(0.0702) = 0.184$$

$$q = np + (n - 1)p'\frac{d'}{d} = 0.1205 + \frac{9}{10}(0.0702)\frac{2.25}{41.5} = 0.124$$

k = 0.35 (table 11 RCDH of ACI)

$$kd = 0.35(41.5) = 14.5 \text{ in.}$$

$$I_t = \frac{20(14.5)^3}{3} + 9(6)(12.25)^2 + 10(10)(27.0)^2$$

$$= 20,300 + 8100 + 73,000 = 101,400 \text{ in.}^4$$

$$I_1 = 0.5(I_g + I_t) = 126,000 \text{ in.}^4$$

Tee section at midspan

The tension steel at midspan = 0.6(10) = 6 bars (par. 7-43d)

$np = 10(6)/82\,(42.75) = 0.0171$

$np' = 10(3)/82\,(42.75) = 0.00855$

$m = np + (n - 1)p' = 0.0171 + 0.9(0.00855) = 0.0248$

$$q = np + (n - 1)p'\frac{d'}{d} = 0.0171 + 0.9(0.00855)\frac{2.25}{42.75} = 0.0175$$

$k = 0.165$ (table 11 RCDH of ACI)

$kd = 0.165(42.75) = 7.05$ in.

$I_t = 81,300 \text{ in.}^4$, $I_g = 259,200 \text{ in.}^4$, $I_a = (I_g + I_t)0.5 = 170,200 \text{ in.}^4$

$I_1/I_2 = 126,700/170,200 = 0.74$

$f_1 = 8.8$ (fig. 6.27)

$f_2 = 0.655$ (fig. 6.28)

$f_3 = 232$ (fig. 6.29)

$$k_1 = \frac{f_3 EI_1}{L^3} = \frac{(232)3(10)^3 126,700}{(20)^3 144} = 79,000 \text{ kips/ft}$$

$$T_n = 2\pi\sqrt{\frac{K_{LM}m}{k_1}} = 6.28\sqrt{\frac{0.78(1.69)}{79,000}} = 0.0257 \text{ sec}$$

$T_r/T_n = 0.032/0.0257 = 1.25$

D.L.F. = 1.18 (fig. 5.21)

Required R_m = 1.18(24.4)(20) = 575 kips

f. Preliminary Design for Bond Stress.

Estimated $V_{max} = 0.62R_m = 0.62(575) = 356$ kips

Allowable $u = 0.15f'_c = 0.15(3000) = 450$ psi (par. 4-09)

$$\Sigma o = \frac{V}{ujd} = \frac{8(356,000)}{450(7)41.5} = 22.0 \text{ in.}$$

A_s = 10 #9 bars in two rows, $A_s = 10 \text{ in.}^2$

$$\Sigma o = 35.4 \text{ in.}, \quad p = A_s/bd = \frac{10}{20(41.5)} = 0.01205$$

g. Determination of Maximum Deflection and Dynamic Reactions by Numerical Integration. Since the trial size may be used directly without modification reference is made to the previous computations for pertinent data.

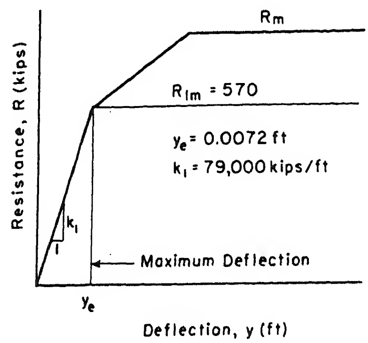


Figure 7.66. Resistance function for girder spanning 20 ft

Revised girder weight $= \frac{20(37.25)(20)150}{12(12)1000}$

$= 15.6$ kips

Total mass $= \frac{41.0 + 15.6}{32.2} = 1.76$ kip-sec^2/ft

Static load girder moment $= \frac{WL}{8}$

$= \frac{(41.0 + 15.6)20}{8} = 142$ kip-ft

Maximum girder resistance in elastic range $R_{1m} = \frac{8.8(1635 - 142 - 197)}{20}$

$= 570$ kips

$$y_e = \frac{R_{1m}}{k_1} = \frac{570}{79,000} = 0.0072 \text{ ft}$$

The basic equation for the numerical integration in table 7.20 is $y_{n+1} = \ddot{y}_n(\Delta t)^2 + 2y_n - y_{n-1}$ (table 5.3) where

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(\Delta t)^2}{K_{IM}(m)} = \frac{(P_n - R_n)(0.0025)^2}{K_{IM}(m)}$$

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(6.25)(10^{-6})}{0.78(1.76)} = 4.55\,(10^{-6})(P_n - R_n) \text{ ft, elastic range}$$

The time interval $\Delta t = 0.0025$ sec is approximately $T_n/10 = 0.00257$ sec (par. 5-08).

The dynamic reaction equations are listed in paragraph 7-43b. The P_n values for the second column are obtained from figure 7.65, multiplying by 2 x 20 to account for the 20-ft span and the two slabs loading the girder.

The maximum deflection computed in table 7.26 is $(y_n)_{max} = 0.0069$ ft. This is less than, but close to, the specified maximum deflection, $y_e = 0.0072$ ft. The design is satisfactory.

Table 7.26. Determination of Maximum Deflection and Dynamic Reactions for Roof Girder

t (sec)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)	V_{1n} (kips)	V_{2n} (kips)
0	0	0	4.4	0.000020	0	0	0
0.0025	28.0	1.6	26.4	0.000120	0.000020	4	6
0.0050	56.0	12.6	43.4	0.000197	0.000160	10	16
0.0075	96.0	39.3	56.7	0.000258	0.000497	22	35
0.0100	136.0	86.3	49.7	0.000226	0.001092	39	63
0.0125	192.0	151.1	40.9	0.000186	0.001913	62	102
0.0150	256.0	230.7	25.3	0.000115	0.002920	91	148
0.0175	312.0	319.3	-7.3	-0.000033	0.004042	120	197
0.0200	360.0	402.2	-42.2	-0.000192	0.005091	148	241
0.0225	408.0	469.9	-61.9	-0.000282	0.005948	171	280
0.0250	436.0	515.3	-79.3	-0.000361	0.006523	186	304
0.0275	460.0	532.2	-72.2	-0.000328	0.006737	194	316
0.0300	472.0	523.2	-51.2	-0.000233	0.006623	193	315
0.0325	480.0	495.8	-15.8	-0.000072	0.006276	186	304
0.0350	484.0	462.7	21.3	0.000097	0.005857	178	291
0.0375	488.0	437.3	50.7	0.000231	0.005535	172	281
0.0400	488.0	430.1	57.9	0.000263	0.005444	170	278
0.0425	488.0	443.7	44.3	0.000202	0.005616	174	284
0.0450	488.0	473.2	14.8	0.000067	0.005990	182	296
0.0475	488.0	508.0	-20.0	-0.000091	0.006431	191	311
0.0500	488.0	535.7	-47.7	-0.000217	0.006781	198	323
0.0525	488.0	546.2	-58.2	-0.000265	0.006914	201	328
					0.006782		

* $(y_n)_{max} = 0.0069$ ft.

h. Shear and Bond Strength.

For interior support end of girder (fixed end of idealized girder)

$V_{max} = 328$ kips (table 7.26)

For no shear reinforcement, allowable $v_y = 0.04f'_c + 5000p$ (eq 4.24)

$v_y = 0.04(3000) + 5000(0.01205) = 120 + 61 = 181$ psi

$$v = \frac{8V}{7bd} = \frac{8(328,000)}{7(20)41.5} = 452 \text{ psi}$$

Shear reinforcement required for $452 - 181 = 271$ psi

Contribution of shear reinforcement to allowable shear stress $= rf_y$

$$r = \frac{271}{40,000} = 0.00677$$

Try 4 #4, $A_s = 0.80$ in.^{2}

$$r = \frac{A_s}{bs} = \frac{0.80}{20(s)} = 0.00677, \therefore s = 5.9 \text{ in., use } s = 6 \text{ in.}$$

For exterior support end of girder (pinned end of idealized girder)

$V_{max} = 201$ kips (table 7.26)

$v_y = 0.04(3000) + 5000(0.00171) = 120 + 9 = 129$ psi

$$v = \frac{8V}{7bd} = \frac{8(201,000)}{7(20)(42.75)} = 268 \text{ psi}$$

Shear reinforcement required for 268 - 129 = 139 psi

$$r = \frac{139}{40,000} = 0.00347$$

Try 4 #4, $A_s = 0.80$ in.^{2}

$$r = \frac{A_s}{bs} = \frac{0.80}{20(s)} = 0.00347; \therefore s = 11.5 \text{ in., use } s = 11 \text{ in.}$$

Bond

$$u = \frac{8V}{7\Sigma o d} = \frac{8(201,000)}{7(21.3)42.75} = 252 \text{ psi}$$

Allowable $u = 0.15f'_c = 0.15(3000) = 450$ psi; OK

i. Summary.

Girder 20-in. × 45-in. tee beam

7-44 FINAL DESIGN OF COLUMN. This column design for plastic behavior was begun in paragraph 7-42. The calculations which follow illustrate the steps which are needed to determine the adequacy of the preliminary design. The primary objective is the calculation of the lateral displacement of the top of the column as a function of time by a numerical integration.

In the preliminary design of the column some of the factors which affect the maximum deflection of the column are neglected to simplify the computations. These factors which are now considered are: the variation of plastic hinge moment with direct stress, the variation of column resistance with lateral deflection, the effect of girder flexibility on the stiffness of the frame, the difference between the load on the wall slab, and the dynamic reactions from the wall which are used as the lateral design load for the frame columns.

Reference should be made to the preliminary design in paragraph 7-42.

a. Mass Computation.

Roof slab = 82.0 kips (par. 7-42b)

$$\text{Girder stem} = \frac{34.25(20)(41.67)150}{12(12)1000} = 29.8 \text{ kips}$$

$$\text{Columns} = \frac{12(20)13(150)3}{12(12)1000} = 9.75 \text{ kips}$$

Walls = 74.8 kips (par. 7-42b)

Mass of single-degree-of-freedom system = total roof +

$$\frac{1}{3}\,(\text{columns} + \text{walls}) = \frac{82.0 + 29.8 + 0.33(9.75 + 74.8)}{32.2}$$

$$= 4.35 \text{ kip-sec}^2/\text{ft}$$

b. Column Properties. (See par. 7-42h.)

$b = 12$ in., $t = 20$ in., $d = 17.56$ in.

$d' = 15.12$ in., $d'' = 2.44$ in., $A_s = A_s' = 3$ in.^{2}, $p = 0.0142$

$$M_D = A_s f_{dy} d' + P_D \left[0.5t - \frac{P_D}{1.7bf'_{dc}}\right] \quad \text{(eq 4.32)}$$

$$= \frac{3(52)(15.12)}{12} + \frac{P_D}{12}\left[10 - \frac{P_D}{1.7(12)3.9}\right]$$

$$M_D = 196.6 + 0.83P_D - 0.00105P_D^2$$

c. Effect of Girder Flexibility. Consideration of the relative flexibility of the girders generally results in a value of the frame elastic spring constant k which is less than the value obtained for infinitely stiff girders in the preliminary design (par. 7-08). To obtain this revised value a simple sidesway analysis of the frame is made. From the sidesway analysis k is the magnitude of the lateral load required to cause unit displacement.

The elastic sidesway analysis (the results of which are shown in figure 7.67) is performed for initial column moments of -1000 kip-ft at top and bottom of each column. This is equivalent to a lateral displacement of the top of the column.

$$x = \frac{(\text{F.E.M.})h^2}{6EI} = \frac{1000(14.63)^2 144}{6(3)10^3(6890)} = 0.249 \text{ ft}$$

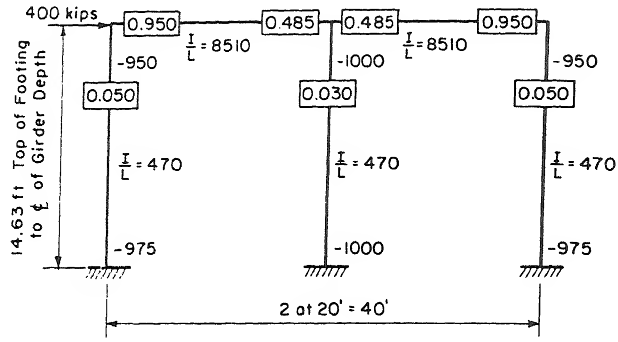


Figure 7.67. Sidesway analysis by moment distribution

From figure 7.67

$$R = \frac{\Sigma M}{h} = \frac{2(950 + 975 + 1000)}{14.63}$$

$$= 400 \text{ kips}$$

$$k = \frac{R}{x} = \frac{400}{0.249}$$

$$= 1600 \text{ kips/ft}$$

In this example the girders are relatively stiff and there is no significant reduction in spring constants (par. 7-42e).

d. Loading. The F_n column of the numerical integration analysis (table 7.27) is obtained from figure 7.68. The first part of figure 7.68

Table 7.27. Determination of Column Adequacy

t (sec)	$\overline{P}_{roof}$ (psi)	$\frac{P_n}{3} = (P_D)_n$ (kips)	P_n/h_c (kips/ft)	M_D (kips/ft)	R_m (kips)	F_n (kips)	R_n (kips)	$\frac{P_n}{h_c} x_n$ (kips)	$F_n - R_n + \frac{P_n}{h_c} x_n$ (kips)	$\ddot{x}_n(\Delta t)^2$ (ft)	x_n (ft)
0	0	37.3	8.8			125	0	0	62.5	0.00144	0
0.01	2.90	148.1	34.9			355	2.3	0.1	352.8	0.00811	0.00144
0.02	5.80	259.0	60.9			386	17.6	0.7	369.1	0.00849	0.01099
0.03	8.70	369.9	87.0			366	46.4	2.5	322.1	0.00740	0.02903
0.04	8.30	354.6	83.4	358.9	168.9	355	87.2	4.5	272.3	0.00626	0.05447
0.05	7.90	339.3	79.8	357.3	168.1	282	137.9	6.9	151.0	0.00347	0.08617
0.06	7.51	324.4	76.3	355.4	167.2	145	167.2	9.3	-12.9	-0.00030	0.12134
0.07	7.15	310.6	73.1	353.1	166.2	100	166.2	11.4	-54.8	-0.00126	0.15621
0.08	6.84	298.8	70.3	350.9	165.1	55	165.1	13.3	-96.8	-0.00222	0.18982
0.09	6.53	286.9	67.5	348.3	163.9	54	163.9	14.9	-95.0	-0.00218	0.22121
0.10	6.26	276.6	65.1	345.9	162.8	52	162.8	16.3	-94.5	-0.00217	0.25042
0.11	6.00	266.7	62.7	343.3	161.6	51	161.6	17.4	-93.2	-0.00214	0.27746
0.12	5.76	257.5	60.6	340.7	160.3	50	160.3	18.3	-92.0	-0.00211	0.30236
0.13	5.52	248.3	58.4	338.0	159.1	49	159.1	19.0	-91.1	-0.00209	0.32515
0.14	5.30	239.9	56.4	335.3	157.8	47	157.8	19.5	-91.3	-0.00210	0.34585
0.15	5.11	232.6	54.7	332.9	156.7	46	156.7	19.9	-90.8	-0.00209	0.36445
0.16	4.92	225.4	53.0	330.4	155.5	45	155.5	20.2	-90.3	-0.00208	0.38096
0.17	4.74	218.5	51.4	327.9	154.3	44	154.3	20.3	-90.0	-0.00207	0.39539
0.18	4.58	212.4	50.0	325.5	153.2	43	153.2	20.4	-89.8	-0.00206	0.40775
0.19	4.44	207.0	48.7	323.4	152.2	41	152.2	20.4	-90.8	-0.00209	0.41805
0.20	4.31	202.1	47.5	321.4	151.2	40	151.2	20.2	-91.0	-0.00209	0.42626
0.21	4.18	197.1	46.4	319.4	150.3	38	150.3	20.1	-92.2	-0.00212	0.43238
0.22	4.06	192.5	45.3	317.5	149.4	36	149.4	19.8	-93.6	-0.00215	0.43638
0.23	3.94	187.9	44.2	315.5	148.5	34	148.5	19.4	-95.1	-0.00219	0.43823*
0.24	3.81	182.9	43.0	313.3	147.4	32	147.4				0.43789
0.25	3.70	178.7	42.0	311.4	146.5	30	146.5				

* x_{max} = 0.44 ft.

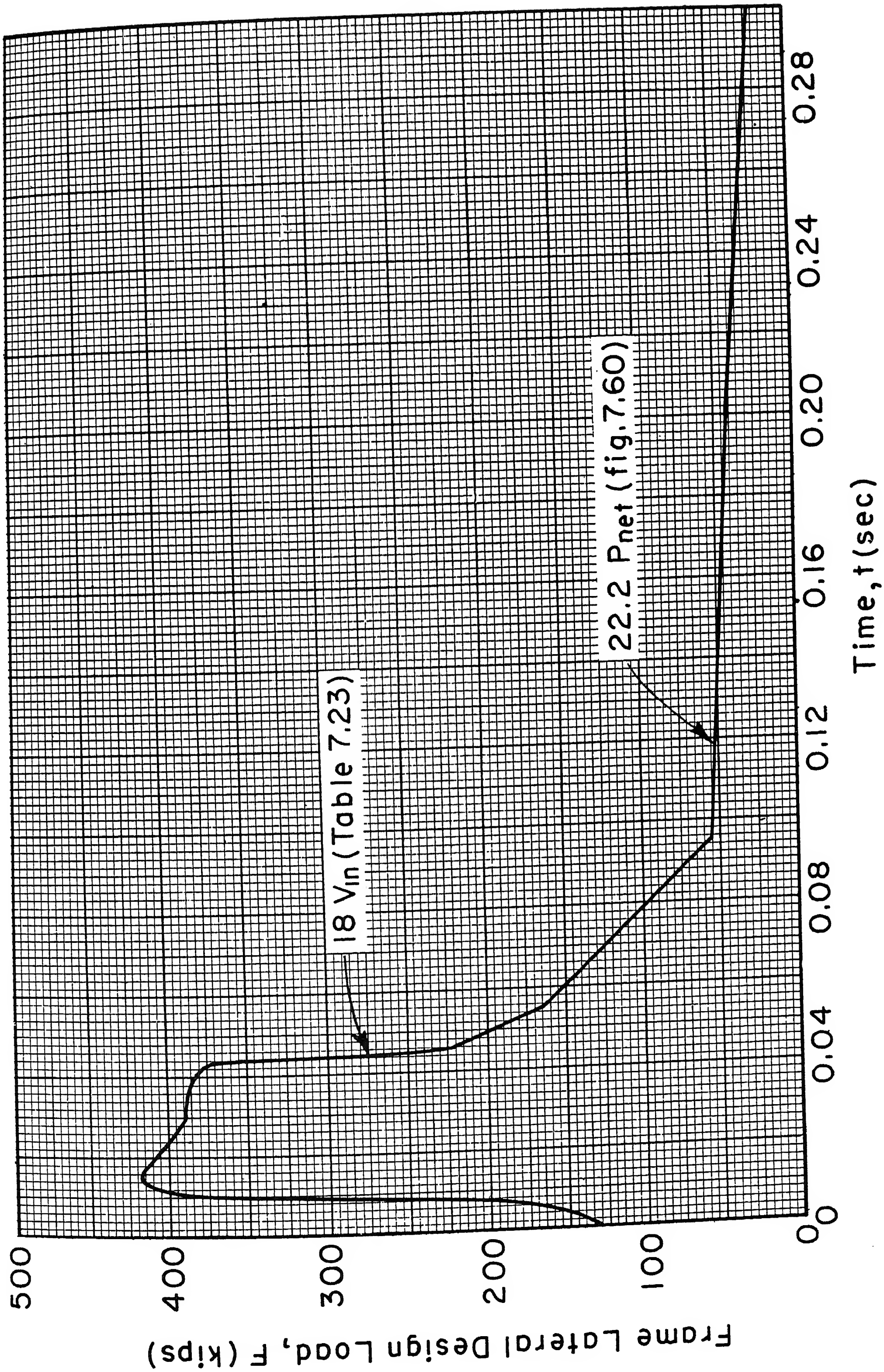


Figure 7.68. Frame lateral design load

is based on V_{1n} dynamic reaction column of the wall slab analysis (table 7.23). The dynamic reactions for a one-foot width of wall are multiplied by 18, the width of one frame bay.

The portion of the curve after t = 0.045 sec is based on the net lateral overpressure curve (fig. 7.60). Here the values of F_n are obtained from

$$F_n = \frac{144(18)8.56}{1000} \overline{P}_{net} = 22.2\overline{P}_{net} \text{ kips}$$

where $\overline{P}_{net}$ is in psi. The dimension 8.56 is equal to one-half the front wall clear span plus the roof slab thickness, 1/2 (15.82) + 0.65 = 8.56 ft.

e. Numerical Integration Computation to Determine Column Adequacy. The total vertical load P_n is obtained by multiplying the average roof overpressure (fig. 7.61) by

$$\frac{(44.25)(18)144}{1000} = 115$$

and adding the dead weight of the roof system (112 kips). The $(P_D)_n$ values are the average axial column loads and are obtained by dividing the total vertical load by 3, the number of columns. The $(P_D)_n$ column is used in the formula of paragraph 7-44b to obtain the value of $(M_D)_n$. The value of $(M_D)_n$ is used in turn to obtain the maximum resistance at any time from the relation

$$(R_m)_n = \frac{2n(M_D)_n}{h_c} = \frac{2(3)(M_D)_n}{12.75} = 0.47(M_D)_n$$

R_n is equal to $kx_n = 1600x_n$ in the elastic range. In the plastic range the limiting value of R is R_m. The expression $(P_n x_n)/h_c$ indicates the decrease in resistance corresponding to the increase in moment resulting from the eccentric loading.

The basic equation for the numerical integration analysis in table 7.21 is

$$x_{n+1} = \ddot{x}_n(\Delta t)^2 + 2x_n - x_{n-1} \text{ (table 5.3)}$$

where

$$\ddot{x}_n(\Delta t)^2 = \frac{\left[F_n - R_n + \frac{P_n x_n}{h_c}\right]}{m} (\Delta t)^2 = \frac{\left[F_n - R_n + \frac{P_n x_n}{h_c}\right]}{4.35} (0.01)^2$$

$$= 2.3(10^{-5}) \left[F_n - R_n + \frac{P_n}{h_c} (x_n)\right]$$

The time interval Δt = 0.01 sec used in table 7.27 is less than one-th the natural period, $T_n = 2\pi\sqrt{m/k} = 6.28\sqrt{4.35/1600} = 0.327$ sec. $= 0.01 < \frac{0.327}{10} = 0.0327$ (par. 5-08) in order to provide a more faithful resentation of the load curve (fig. 7.68). The allowable maximum placement $= \frac{\alpha\beta\ R_m}{k} = \frac{6(167.2)}{1600} = 0.627$ ft (par. 7-42c and table 7.21). The puted maximum displacement = 0.44. Thus the design $\alpha\beta = 6\left(\frac{0.44}{0.627}\right) = 4.2$. design is satisfactory.

f. Shear and Bond Stress.

$V_{max} = \frac{167.2}{3} = 55.7$ kips (table 7.27)

For no shear reinforcement

Allowable $v_y = 0.04f'_c + 5000p$ (eq 4.24)

$v_y = 0.04(3000) + 5000(0.0142) = 120 + 71 = 191$ psi

$v = \frac{8V}{7bd} = \frac{8(55,700)}{7(12)(17.56)} = 303$ psi

Shear reinforcement required for 303 - 191 = 112 psi

Try 1 #4, $A_s = 0.20$ in.^{2}

$r = \frac{112}{40,000} = 0.0028$

$r = \frac{A_s}{bs} = 0.0028 = \frac{0.20}{12(s)}$; $\therefore$ s = 5.95 in., use s = 6 in.

$\Sigma o = 8.9$ in.

$u = \frac{8(55,700)}{7(8.9)17.56} = 407$ psi

Allowable $u = 0.15f'_c = 0.15(3000) = 450$ psi > 407 psi; OK

NUMERICAL EXAMPLE, DESIGN OF A ONE-STORY REINFORCED-CONCRETE FRAME BUILDING--ELASTIC AND ELASTO-PLASTIC BEHAVIOR

GENERAL. This numerical example presents the design of a typical bay windowless, one-story, reinforced-concrete rigid-frame building in

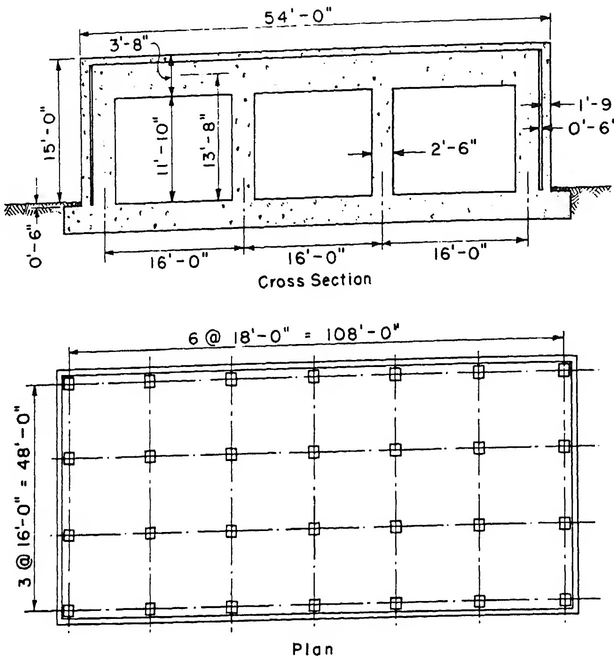


Figure 7.69. Plan and section of building

which the primary elements are permitted to deflect elastically and the others to deflect into the elasto-plastic region. The example includes the design of a typical wall slab, roof slab, girder, and column.

The building consists of reinforced-concrete rigid frames and reinforced-concrete wall and roof slabs. The wall slab is a one-way slab spanning vertically, supported at the bottom by the wall footing which is integral with the foundation, and at the top by the roof slab. The roof slab is also a one-way slab spanning continuously over the reinforced-concrete frames. The roof girders are tee beams formed by the roof slab and a rectangular girder stem. The columns are rectangular tied columns symmetrically reinforced in the strong direction.

7-46 DESIGN PROCEDURE. The over-all design procedure illustrated by this example is essentially the same as the procedure detailed in paragraph 7-29 for the elastic design of a steel rigid-frame building with reinforced concrete walls and roof.

7-47 LOAD DETERMINATION. The computation of loads is explained in EM 1110-345-413 and illustrated again in paragraph 7-19 for a one-story building. In this example the necessary load curves are presented without explanation or computation. The design overpressure of 10 psi is selected arbitrarily for this example.

The overpressure vs time curves that are presented are:

(1) Incident overpressure vs time (fig. 7.70)
(2) Front face overpressure vs time (fig. 7.71)
(3) Rear face overpressure vs time (fig. 7.72)
(4) Net lateral overpressure vs time (fig. 7.73)
(5) Average roof overpressure vs time (fig. 7.74)

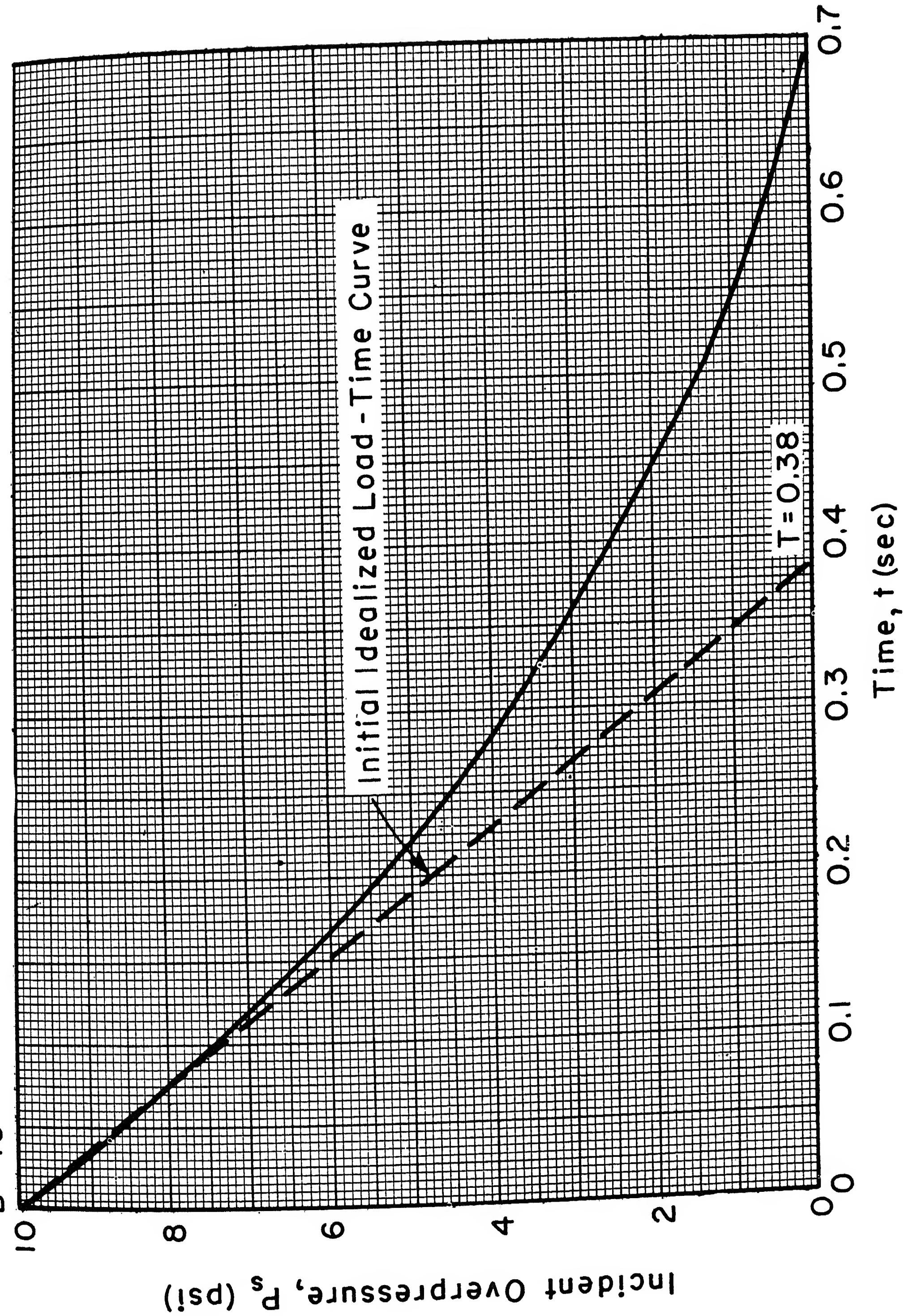


Figure 7.70. Incident overpressure vs time curve

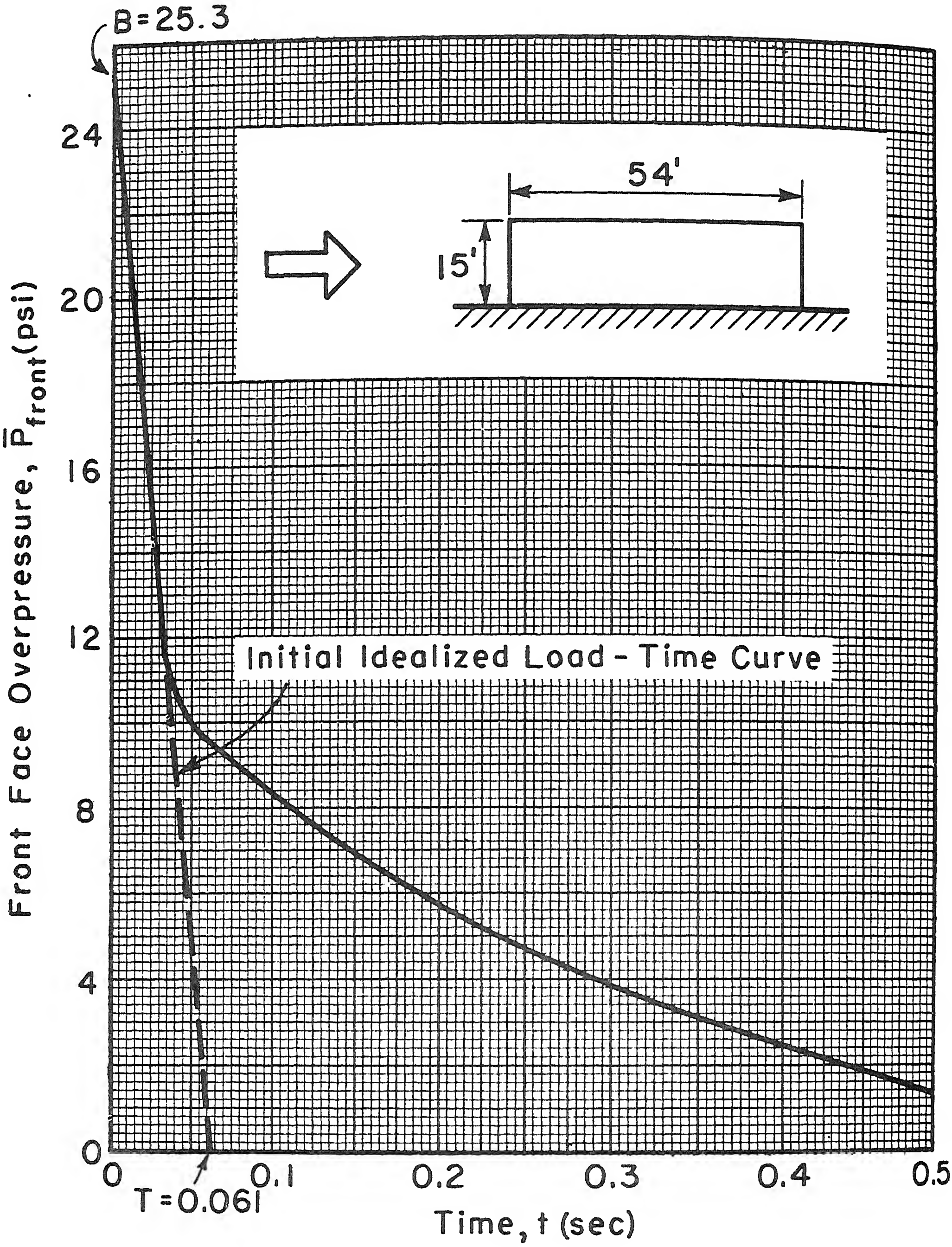


Figure 7.71. Front face overpressure vs time curve

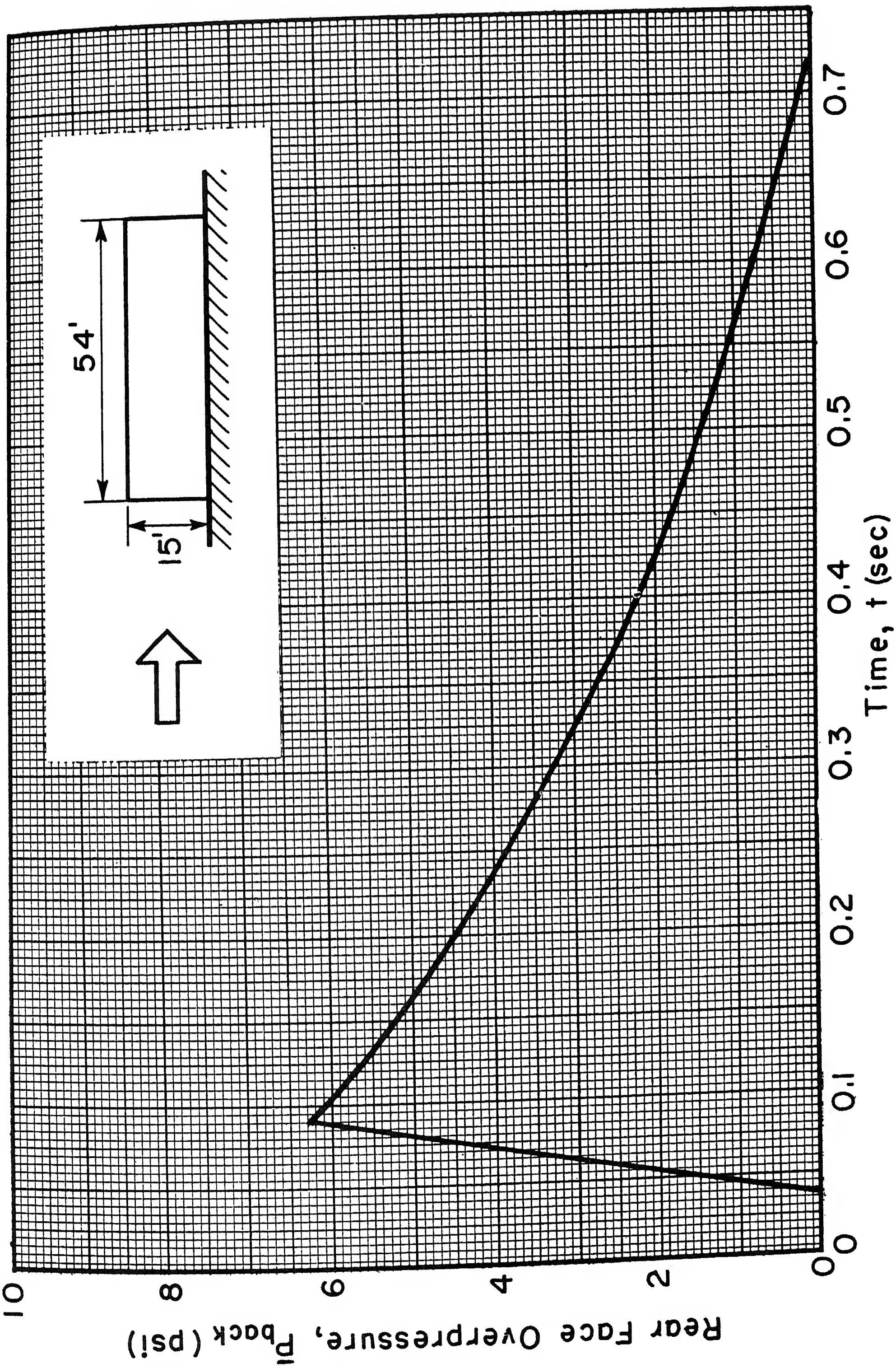


Figure 7.72. Rear face overpressure vs time curve

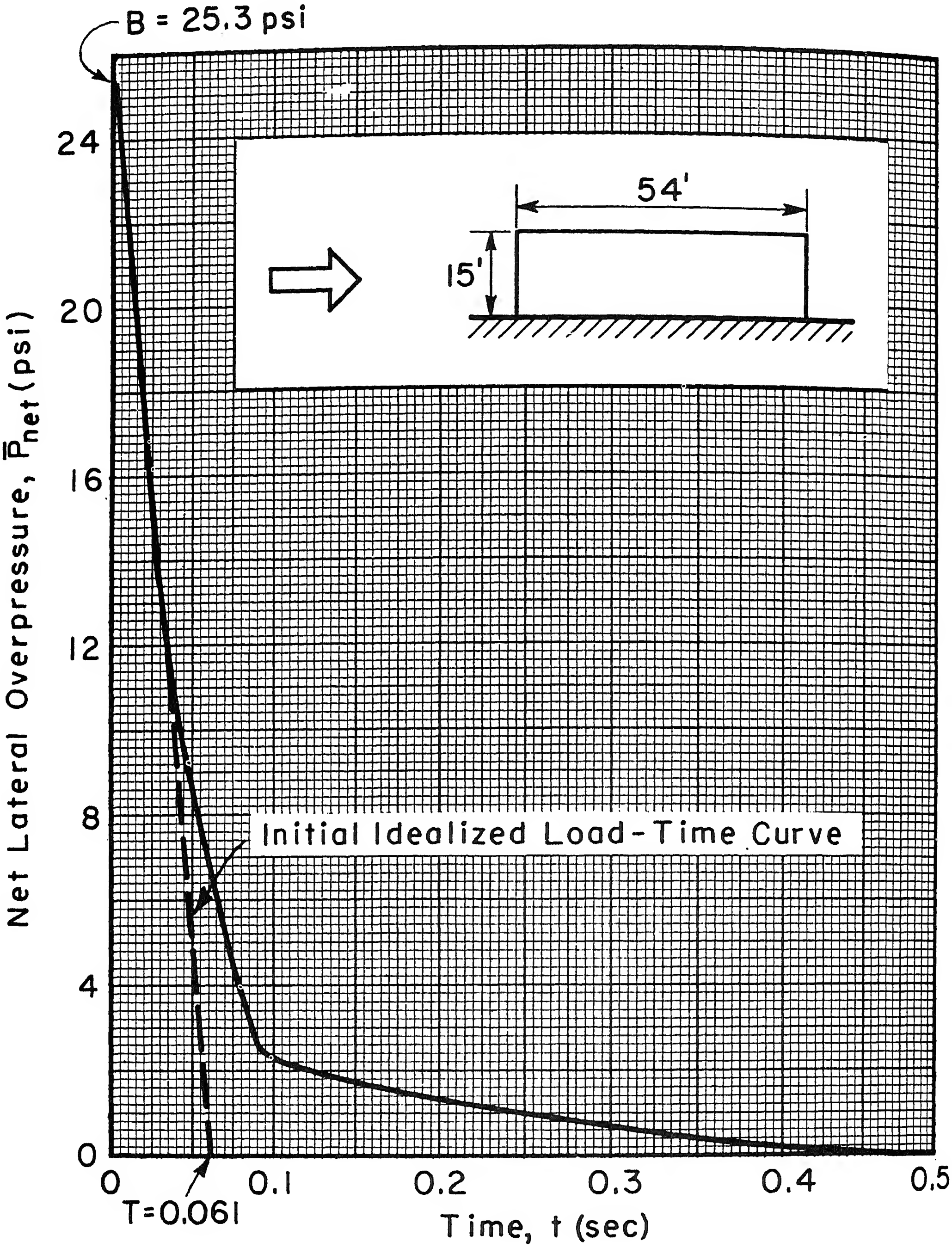


Figure 7.73. Net lateral overpressure vs time curve

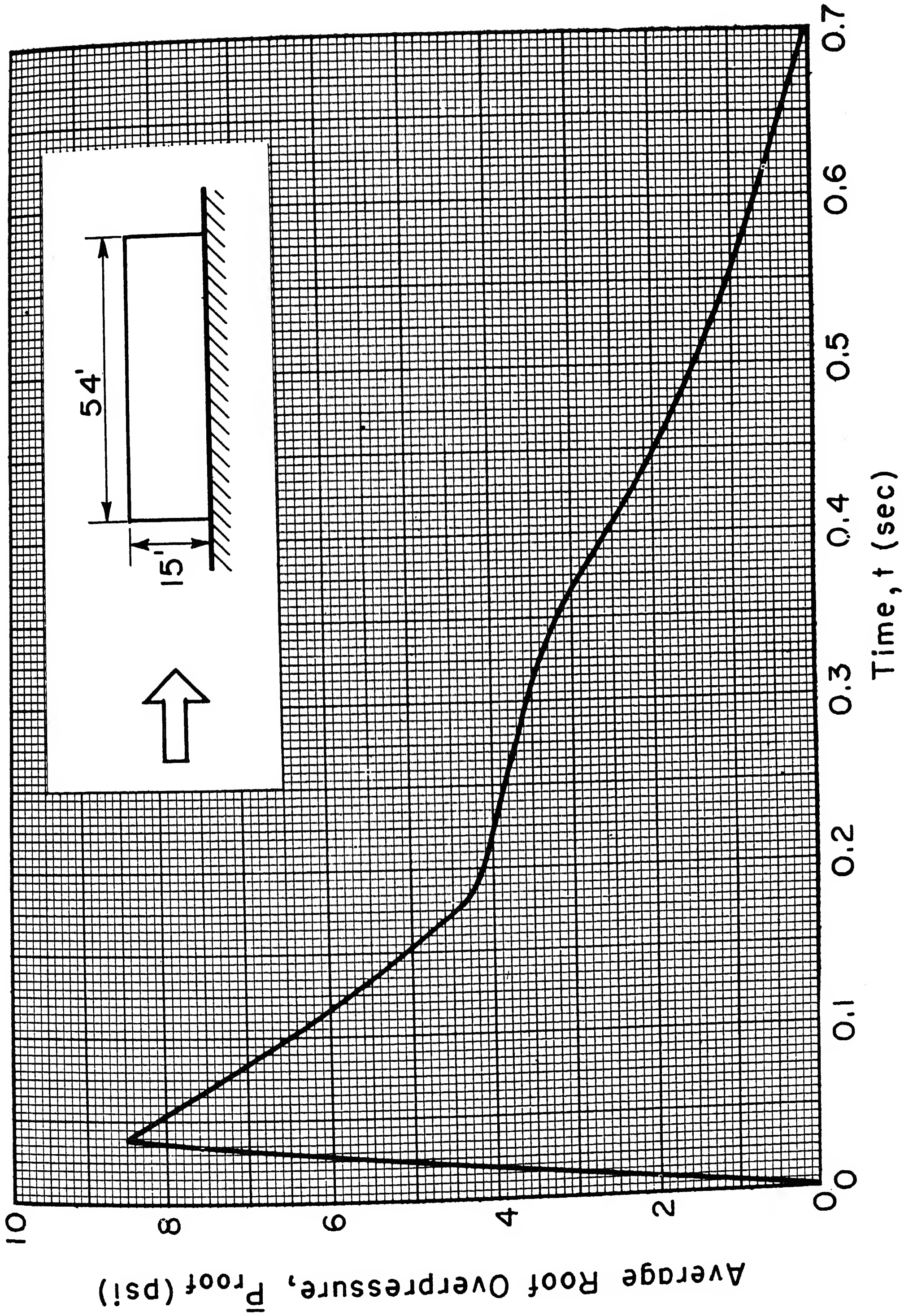
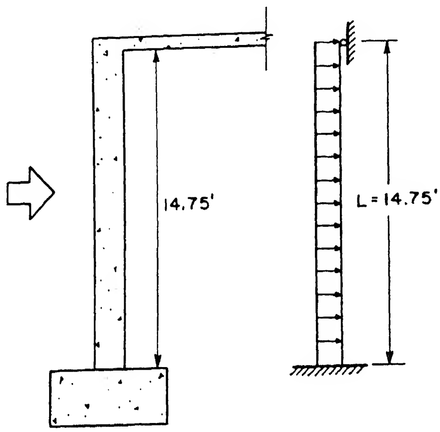


Figure 7.74. Average roof overpressure vs time curve

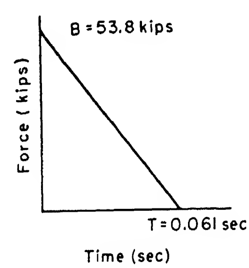
7-48 DESIGN OF WALL SLAB. The wall slab is designed by considering an element one foot wide, spanning vertically between a fixed support at the foundation and a pinned support at the roof slab. The wall slab is designed to deflect through the elasto-plastic range up to the beginning of the plastic range. This means that the design load is expected to cause development of plastic hinges first at the fixed support and then near midspan.



This is an example of a design for which the elasto-plastic design procedure is used. Another method of design which could be used utilizes the elastic design procedure and an artificial maximum resistance (par. 6-14e).

The design span length is equal to the clear distance between the foundation and the roof slab. The dead load stresses are not considered in a vertical wall.

a. Design Loading. The design load as idealized from the computed loading shown by figure 7.71 is defined by:



$$B = 25.3 \text{ psi} = \frac{25.3(144)14.75}{1000} = 53.8 \text{ kips}$$

$$T = 0.061 \text{ sec}$$

$$H = \frac{BT}{2} = \frac{(53.8)0.061}{2} = 1.64 \text{ kip-sec (par. 6-11)}$$

b. Dynamic Design Factors. (Refer to table 6.1.)

Elastic range:

$K_L = 0.58,$ $\quad K_M = 0.45,$ $\quad K_{LM} = 0.78$

$R_{1m} = \dfrac{8M_{Ps}}{L},$ $\quad k_1 = \dfrac{185EI}{L^3}$

$V_1 = 0.26R + 0.12P,$ $\quad V_2 = 0.43R + 0.19P$

Elasto-plastic range:

$K_L = 0.64,$ $\quad K_M = 0.50,$ $\quad K_{LM} = 0.78$

$R_{1m} = \frac{4}{L}(M_{Ps} + 2M_{Pm})$, $\quad k_{ep} = \frac{384EI}{5L^3}$

$V = 0.39R + 0.11P$

lastic range:

$K_{LM} = 0.66$

erage values:

$K_L = 0.5(0.58 + 0.64) = 0.61$

$K_M = 0.5(0.45 + 0.50) = 0.47$

$R_m = \frac{4}{L}(M_{Ps} + 2M_{Pm})$

$k_E = \frac{160EI}{L^3}$ (plastic design, $M_{Ps} = M_{Pm}$)

c. First Trial - Actual Properties.

Let $M_{Ps} = M_{Pm} = M_P$

Assume $p = 0.015$ at midspan cross section (par. 4-10)

Assume $C_R = 1.7$ (experience)

$R_m = C_R B = 1.7(53.8) = 91.5$ kips

$$M_P = pf_{dy}bd^2\left(1 - \frac{pf_{dy}}{1.7f'_{dc}}\right) \quad \text{(eq 4.16)}$$

$$= 0.015(52)(1)d^2\left[1 - \frac{0.015(52)}{1.7(3.9)}\right] = 0.688d^2 \text{ kip-ft (d in inches)}$$

$$R_m = \frac{12M_P}{L} = \frac{(12)0.688d^2}{14.75} = 91.5, \therefore d = 12.75 \text{ in.}$$

Try $h = 14$ in., $d = 12.5$ in., $p = 0.015$, $np = 0.15$

$M_P = 0.688(12.5)^2 = 107.5$ kip-ft

$$R_m = \frac{12M_P}{L} = \frac{12(107.5)}{14.75} = 87.5 \text{ kips}$$

$I_g = bh^3/12 = (14.0)^3 = 2740 \text{ in.}^4$

$$I_t = bd^3\left[\frac{k^3}{3} + np(1 - k)^2\right] = 12(d)^3\left[\frac{(0.42)^3}{3} + 0.15(1 - 0.42)^2\right]$$

$$= 0.905d^3 = 0.905(12.5)^3 = 1770 \text{ in.}^4$$

$I_a = 0.5(I_g + I_t) = 0.5(2740 + 1770) = 2255 \text{ in.}^4$

$$k_E = \frac{160EI}{L^3} = \frac{(160)3(10)^3 2255}{(14.75)^3 144} = 2340 \text{ kips/ft}$$

$$y_E = \frac{R_m}{k_E} = \frac{87.5}{2340} = 0.0374 \text{ ft}$$

$$\text{Weight} = \frac{14(150)(14.75)}{(12)1000} = 2.58 \text{ kips}$$

$$\text{Mass } m = \frac{2.58}{32.2} = 0.08 \text{ kip-sec}^2/\text{ft}$$

$$k_1 = \frac{185}{160} k_E = 1.155(2340) = 2700 \text{ kips/ft}$$

$$k_{ep} = \frac{384}{5(160)} k_E = 0.48(2340) = 1120 \text{ kips/ft}$$

$$R_{1m} = \frac{8M_P}{L} = \frac{8(107.5)}{14.75} = 58.3 \text{ kips}$$

$$y_e = \frac{R_{1m}}{k_1} = \frac{58.3}{2700} = 0.0216 \text{ ft}$$

$$y_m = y_{ep} = y_e + \frac{R_m - R_{1m}}{k_{ep}} = 0.0216 + \frac{87.5 - 58.3}{1120} = 0.0476 \text{ ft}$$

d. First Trial - Equivalent System Properties.

$R_{me} = K_L R_m = 0.61(87.5) = 53.4$ kips (eq 6.12)

$H_e = K_L H = 0.61(1.64) = 1.0$ kip-sec (eq 6.8)

$m_e = K_M m = 0.47(0.08) = 0.0376$ kip-sec^2/ft (eq 6.2)

$$T_n = 2\pi\sqrt{\frac{K_{LM} m}{k_E}} = 6.28\sqrt{\frac{0.78(0.08)}{2340}} = 0.0324 \text{ sec}$$

$$W_P = \frac{(H_e)^2}{2m_e} = \frac{(1.0)^2}{2(0.0376)} = 13.3 \text{ ft-kips (eq 6.10)}$$

e. First Trial-Work Done vs Energy Absorption Capacity.

$C_T = T/T_n = 0.061/0.0324 = 1.88$

$C_R = R_m/B = 87.5/53.8 = 1.63$ (eqs 6.15, 6.16)

$t_m/T = 0.26$ (fig. 5.29)

$t_m = (0.26)0.061 = 0.016$ sec

Idealized load-time curve is satisfactory (fig. 7.71)(par. 5-13)

$C_W = 0.091$ (fig. 5.27)

$W_m = C_W W_P = 0.091(13.3) = 1.21$ ft-kips (eq 6.17)

$$E = R_{me}(y_m - 0.5y_e) = 53.4\ [0.0476 - 0.5(0.0216)]$$

$$= 1.96 \text{ ft-kips (eq 6.18)}$$

$E \gg W$, therefore the selected proportions are satisfactory as a eliminary design.

f. Second Trial - Actual Properties.

$$R_m = \frac{0.5(W_m + E)}{K_L(y_m - 0.5y_e)} = \frac{0.5(1.96 + 1.2)}{(0.61)\ [0.0476 - 0.5(0.0216)]} = 70.5 \text{ kips (eq 6.19)}$$

$$R_m = \frac{12M_P}{L} = \frac{(12)0.688d^2}{14.75} = 70.5, \ \therefore\ d = 11.2 \text{ in.}$$

Try h = 12.5 in., d = 11.0 in., p = 0.015

$$M_P = 0.688d^2 = 0.688(11)^2 = 83.3 \text{ kip-ft}$$

$$R_m = \frac{12M_P}{L} = \frac{12(83.3)}{14.75} = 68 \text{ kips}$$

$$I_g = \frac{bh^3}{12} = (12.5)^3 = 1950 \text{ in.}^4$$

$$I_t = 0.905d^3 = 0.905(11.0)^3 = 1210 \text{ in.}^4 \ (k = 0.42)$$

$$I_a = 0.5(I_g + I_t) = 0.5(1950 + 1210) = 1580 \text{ in.}^4$$

$$k_E = \frac{160EI}{L^3} = \frac{(160)3(10)^3 1580}{(14.75)^3 144} = 1640 \text{ kips/ft}$$

$$y_E = \frac{R_m}{k_E} = \frac{68}{1640} = 0.0415 \text{ ft}$$

$$\text{Weight} = \frac{12.5(150)(14.75)}{(12)1000} = 2.3 \text{ kips}$$

$$\text{Mass} \quad m = \frac{2.3}{32.2} = 0.0715 \text{ kip-sec}^2/\text{ft}$$

$$k_1 = \frac{185}{160} k_E = 1.155(1640) = 1890 \text{ kips/ft}$$

$$k_{ep} = \frac{384}{5(160)} k_E = 0.48(1640) = 788 \text{ kips/ft}$$

$$R_{1m} = \frac{8M_P}{L} = \frac{8(83.3)}{14.75} = 45.0 \text{ kips}$$

$$y_e = \frac{R_{1m}}{k_1} = \frac{45}{1890} = 0.0238 \text{ ft}$$

$$y_m = y_{ep} = y_e + \frac{R_m - R_{1m}}{k_{ep}} = 0.0238 + \frac{68 - 45.0}{788} = 0.053 \text{ ft}$$

g. Second Trial - Equivalent System Properties.

$R_{me} = K_L R_m = 0.61(68) = 41.5$ kips (eq 6.12)

$H_e = K_L H = 0.61(1.64) = 1.0$ kip-sec (eq 6.8)

$m_e = K_M m = 0.47(0.0715) = 0.0336$ kip-sec^2/ft (eq 6.2)

$$T_n = 2\pi\sqrt{\frac{K_{LM} m}{k_E}} = 6.28\sqrt{\frac{0.78(0.0715)}{1640}} = 0.0366 \text{ sec}$$

$$W_P = \frac{(H_e)^2}{2m_e} = \frac{(1.0)^2}{2(0.0336)} = 14.9 \text{ ft-kips (eq 6.10)}$$

h. Work Done vs Energy Absorption Capacity.

$C_T = T/T_n = 0.061/0.0366 = 1.67$

$C_R = R_m/B = 68/53.8 = 1.26$ (eqs 6.15, 6.16)

$t_m/T = 0.34$ (fig. 5.29)

$t_m = (0.34)0.061 = 0.0207$ sec

Idealized load-time curve is satisfactory (fig. 7.71)

$C_W = 0.118$ (fig. 5.27)

$W_m = C_W W_P = 0.118(14.9) = 1.76$ ft-kips (eq 6.17)

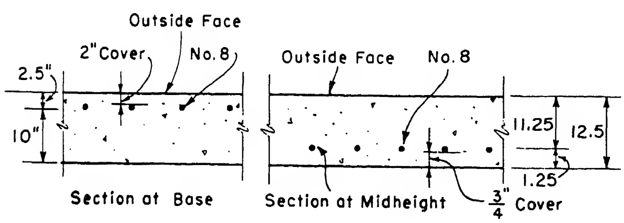
$E = R_{me}(y_m - 0.5y_e) = 41.5\ [0.053 - 0.5(0.0238)] = 1.71$ ft-kips (eq 6.18)

$E \approx W$, therefore the selected proportions are not satisfactory as a preliminary design.

i. Preliminary Design for Bond Stress.

At bottom of wall (at fixed end of idealized slab)

This section has smaller d than at midspan therefore $p > 0.015$. From equation 4.16 for d = 10 in. and $M_P = 83.3$ kip-ft, required $p \approx 0.0184$.



Estimated $V_{max} = 1/2\ R_m$

$= 1/2\ (68) = 34$ kips

Allowable $u = 0.15f'_c$

$= 0.15(3000)$

$= 450$ psi (par. 4-09)

$$\Sigma o = \frac{V}{ujd} = \frac{8(34,000)}{450(7)10} = 8.65 \text{ in.}$$

Try #8 at 4-1/4 in., $A_s = 2.23$ in.^{2}/ft, $\Sigma o = 8.9$ in./ft

$$p = \frac{A_s}{bd} = \frac{2.23}{12(10)} = 0.0186$$

top of wall (at pinned end of idealized slab)

Estimated $V_{max} = 1/3\ R_m = 1/3\ (68) = 23$ kips

$$\Sigma o = \frac{8(23,000)}{450(7)11.25} = 5.2 \text{ in.}$$

Bond stress is not critical

Try #8 at 4-1/4 in., $A_s = 2.23$ in.^{2}, $\Sigma o = 8.9$ in.

$$p = \frac{A_s}{bd} = \frac{2.23}{11.25(12)} = 0.0165$$

j. Determination of Maximum Deflection and Dynamic Reactions by merical Integration.

$$M_{Pm} = pf_{dy}bd^2\left(1 - \frac{pf_{dy}}{1.7f'_c}\right) = 0.0165(52)(1)(11.25)^2\left[1 - \frac{(0.0165)52}{1.7(3.9)}\right]$$

$$= 94.5 \text{ kip-ft (eq 4.16)}$$

$$M_{Ps} = 0.0186(52)1(10)^2\left[1 - \frac{0.0186(52)}{1.7(3.9)}\right] = 82.6 \text{ kip-ft}$$

$$I_g = bh^3/12 = (12.5)^3 = 1950 \text{ in.}^4$$

$$I_t = bd^3\left[\frac{k^3}{3} + np(1 - k)^2\right] = 12(11.25)^3\left[\frac{0.42^3}{3} + 0.148(1 - 0.42)^2\right]$$

$$= 1290 \text{ in.}^4$$

$$I_a = 0.5(I_g + I_t) = 0.5(1950 + 1290) = 1620 \text{ in.}^4$$

$$\text{Weight} = \frac{12.5(150)14.75}{12(1000)} = 2.3 \text{ kips}$$

$$\text{Mass } m = \frac{2.3}{32.2} = 0.0715 \text{ kip-sec}^2\text{/ft}$$

stic range:

$$R_{1m} = \frac{8M_{Ps}}{L} = \frac{8(82.6)}{14.75} = 44.8 \text{ kips}$$

$$k_1 = \frac{185EI}{L^3} = \frac{(185)3(10)^3 1620}{(14.75)^3 144} = 1940 \text{ kips/ft}$$

$$y_m = y_{ep} = y_e + \frac{R_m - R_{1m}}{k_{ep}} = 0.0238 + \frac{68 - 45.0}{788} = 0.053 \text{ ft}$$

g. <u>Second Trial - Equivalent System Properties.</u>

$R_{me} = K_L R_m = 0.61(68) = 41.5$ kips (eq 6.12)

$H_e = K_L H = 0.61(1.64) = 1.0$ kip-sec (eq 6.8)

$m_e = K_M m = 0.47(0.0715) = 0.0336$ kip-sec^2/ft (eq 6.2)

$$T_n = 2\pi\sqrt{\frac{K_{LM} m}{k_E}} = 6.28\sqrt{\frac{0.78(0.0715)}{1640}} = 0.0366 \text{ sec}$$

$$W_P = \frac{(H_e)^2}{2m_e} = \frac{(1.0)^2}{2(0.0336)} = 14.9 \text{ ft-kips (eq 6.10)}$$

h. <u>Work Done vs Energy Absorption Capacity.</u>

$C_T = T/T_n = 0.061/0.0366 = 1.67$

$C_R = R_m/B = 68/53.8 = 1.26$ (eqs 6.15, 6.16)

$t_m/T = 0.34$ (fig. 5.29)

$t_m = (0.34)0.061 = 0.0207$ sec

Idealized load-time curve is satisfactory (fig. 7.71)

$C_W = 0.118$ (fig. 5.27)

$W_m = C_W W_P = 0.118(14.9) = 1.76$ ft-kips (eq 6.17)

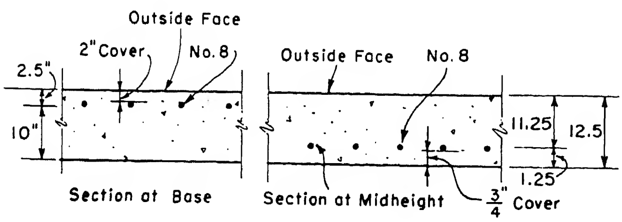
$E = R_{me}(y_m - 0.5y_e) = 41.5\ [0.053 - 0.5(0.0238)] = 1.71$ ft-kips (eq 6.18)

$E \approx W$, therefore the selected proportions are not satisfactory as a preliminary design.

i. <u>Preliminary Design for Bond Stress.</u>

<u>At bottom of wall (at fixed end of idealized slab)</u>

This section has smaller d than at midspan therefore $p > 0.015$. From equation 4.16 for $d = 10$ in. and $M_p = 83.3$ kip-ft, required $p \approx 0.0184$.



Estimated $V_{max} = 1/2\ R_m$

$= 1/2\ (68) = 34$ kips

Allowable $u = 0.15f'_c$

$= 0.15(3000)$

$= 450$ psi (par. 4-09)

$$\Sigma o = \frac{V}{ujd} = \frac{8(34,000)}{450(7)10} = 8.65 \text{ in.}$$

Try #8 at 4-1/4 in., $A_s = 2.23 \text{ in.}^2/\text{ft}$, $\Sigma o = 8.9$ in./ft

$$p = \frac{A_s}{bd} = \frac{2.23}{12(10)} = 0.0186$$

top of wall (at pinned end of idealized slab)

Estimated $V_{max} = 1/3\ R_m = 1/3\ (68) = 23$ kips

$$\Sigma o = \frac{8(23,000)}{450(7)11.25} = 5.2 \text{ in.}$$

Bond stress is not critical

Try #8 at 4-1/4 in., $A_s = 2.23 \text{ in.}^2$, $\Sigma o = 8.9$ in.

$$p = \frac{A_s}{bd} = \frac{2.23}{11.25(12)} = 0.0165$$

j. Determination of Maximum Deflection and Dynamic Reactions by
erical Integration.

$$M_{Pm} = pf_{dy}bd^2\left(1 - \frac{pf_{dy}}{1.7f'_c}\right) = 0.0165(52)(1)(11.25)^2\left[1 - \frac{(0.0165)52}{1.7(3.9)}\right]$$

$$= 94.5 \text{ kip-ft (eq 4.16)}$$

$$M_{Ps} = 0.0186(52)1(10)^2\left[1 - \frac{0.0186(52)}{1.7(3.9)}\right] = 82.6 \text{ kip-ft}$$

$$I_g = bh^3/12 = (12.5)^3 = 1950 \text{ in.}^4$$

$$I_t = bd^3\left[\frac{k^3}{3} + np(1 - k)^2\right] = 12(11.25)^3\left[\frac{0.42^3}{3} + 0.148(1 - 0.42)^2\right]$$

$$= 1290 \text{ in.}^4$$

$$I_a = 0.5(I_g + I_t) = 0.5(1950 + 1290) = 1620 \text{ in.}^4$$

$$\text{Weight} = \frac{12.5(150)14.75}{12(1000)} = 2.3 \text{ kips}$$

$$\text{Mass } m = \frac{2.3}{32.2} = 0.0715 \text{ kip-sec}^2/\text{ft}$$

stic range:

$$R_{1m} = \frac{8M_{Ps}}{L} = \frac{8(82.6)}{14.75} = 44.8 \text{ kips}$$

$$k_1 = \frac{185EI}{L^3} = \frac{(185)3(10)^3 1620}{(14.75)^3 144} = 1940 \text{ kips/ft}$$

$$y_e = \frac{R_{1m}}{k_1} = \frac{44.8}{1940} = 0.023 \text{ ft}$$

Elasto-plastic range:

$$R_m = \frac{4(M_{Ps} + 2M_{Pm})}{L} = \frac{4[82.6 + 2\ (94.5)]}{14.75} = 73.5 \text{ kips}$$

$$k_{ep} = \frac{384EI}{5L^3} = \frac{384}{5(185)} k_1 = 806 \text{ kips/ft}$$

$$y_{ep} = y_e + \frac{R_m - R_{1m}}{k_{ep}} = 0.023 + \frac{73.5 - 44.8}{806} = 0.058 \text{ ft}$$

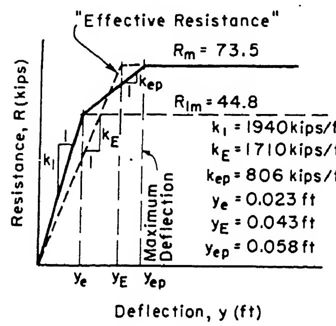


Figure 7.75. Resistance function for 12-1/2-in. slab spanning 14.75 ft, fixed at one support, pinned at other support

By providing equal areas under the "effective resistance" line and the computed elasto-plastic resistance line, $y_E = 0.043$ ft (fig. 7.75)

$$k_E = \frac{R_m}{y_E} = \frac{73.5}{0.043} = 1710 \text{ kips/ft}$$

$$T_n = 2\pi\sqrt{\frac{K_{LM}m}{k_E}} = 6.28\sqrt{\frac{0.78(0.0715)}{1710}}$$

$$= 0.0358 \text{ sec}$$

The basic equation for the numerical integration in table 7.28 is $y_{n+1} = \ddot{y}_n(\Delta t)^2 + 2y_n - y_{n-1}$ (table 5.3) where

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(\Delta t)^2}{K_{LM}(m)}$$

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(0.003)^2}{0.78(0.0715)} = 1.62(10)^{-4}(P_n - R_n) \text{ ft, elastic range}$$

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(0.003)^2}{0.78(0.0715)} = 1.62(10)^{-4}(P_n - R_n) \text{ ft, elasto-plastic range}$$

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(0.003)^2}{0.66(0.0715)} = 1.907(10)^{-4}(P_n - R_n) \text{ ft, plastic range}$$

The time interval $\Delta t = 0.003$ sec is less than $T_n/10 = 0.00358$ sec (par. 5-08). The dynamic reaction equations are listed in paragraph 7-48b. The P_n values for the second column are obtained from figure 7.71, multiplying by $144(14.75)/1000 = 2.12$.

Table 7.28. Determination of Maximum Deflection and Dynamic Reactions for Front Wall Slab

t (sec)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)	V_{1n} (kips)	V_{2n} (kips)
0	53.8	0	26.9	0.00436	0	6.5	10.2
0.003	51.2	8.5	42.7	0.00692	0.00436	8.3	13.4
0.006	48.5	30.3	18.2	0.00295	0.01564	13.7	22.3
0.009	45.9	50.4	-4.5	-0.00073	0.02987	24.7	24.7
0.012	43.2	61.2	-18.0	-0.00292	0.04337	28.7	28.7
0.015	40.6	69.7	-29.1	-0.00471	0.05395	31.7	31.7
0.018	38.0	73.5	-35.5	-0.00677	0.05982*	32.5	32.5
0.021	35.3	73.5	-38.2	-0.00728	0.05892	32.1	32.1
0.024	31.8	59.4			0.05074	19.2	31.5

$*(y_n)_{max} = 0.060$ ft.

In table 7.28 the maximum computed deflection is $(y_n)_{max} = 0.060$ ft. is is slightly more than the specified maximum displacement, $y_{ep} = 0.054$. This is an acceptable difference for blast resistant design.

k. Shear Stress and Bond Strength.

bottom of wall (fixed end of idealized beam)

$V_{max} = 32.5$ kips (table 7.28)

For no shear reinforcement

$$v_y = 0.04f'_c + 5000p = 0.04(3000) + 5000(0.0186) = 120 + 93$$
$$= 213 \text{ psi (eq 4.24)}$$

$$v = \frac{8V}{7bd} = \frac{8(32,500)}{7(12)(10)} = 310 \text{ psi}$$

Shear reinforcement required for 310 - 213 = 97 psi

Contribution of shear reinforcement to allowable shear stress = rf_y

$$r = \frac{97}{40,000} = 0.00242$$

Try 1 #4, $A_s = 0.20$ in.^{2}

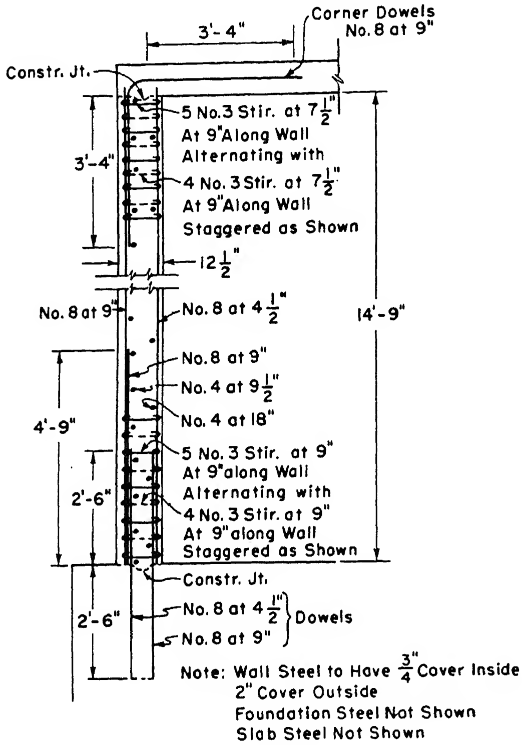
$$r = \frac{A_s}{bs} = \frac{0.20}{8\text{-}1/2\ s} = 0.00242; \therefore\ s = 9.75 \text{ in., use } s = 9 \text{ in.}$$

For top of wall (pinned end of idealized beam)

$$V_{max} = 32.5 \text{ kips (table 7.28)}$$

$$v_y = 0.04f'_c + 5000p = 0.04(3000) + 5000(0.0148) = 120 + 74 = 194 \text{ psi}$$

$$v = \frac{8V}{7bd} = \frac{8(32,500)}{7(12)(11.25)} = 275 \text{ psi}$$



Shear reinforcement required for

$$275 - 194 = 81 \text{ psi}$$

$$r = \frac{81}{40,000} = 0.00202$$

Try 1 #4, $A_s = 0.20 \text{ in.}^2$

$$r = \frac{A_s}{bs} = \frac{0.20}{8\text{-}1/2\ s} = 0.00202;$$

$\therefore$ s = 11.65 in., use s = 11 in.

Bond

$$u = \frac{8V}{7\Sigma o d} = \frac{8(32,500)}{7(8.9)10} = 412 \text{ psi}$$

Allowable $u = 0.15f'_c$

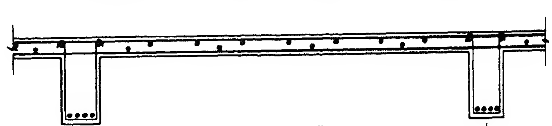
$= 0.15(3000)$

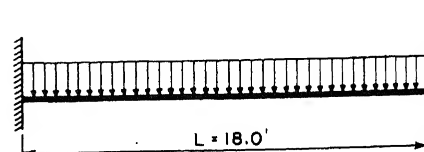
$= 450 \text{ psi} > 412 \text{ psi}$; OK

1. Summary.

12-1/2-in. slab

7-49 DESIGN OF ROOF SLAB. The design procedure followed in designing this roof slab is similar to the design procedure for design of the wall slab in paragraph 7-48. Consideration is given to the static load stresses in this case because they reduce the maximum resistance of the slab. Since the procedures of this manual are limited to single-span elements, the slab is designed by considering the behavior of a one-foot-wide element fixed at both ends and spanning 18 ft between the frame centerlines. The slab is permitted to deflect through the elasto-plastic

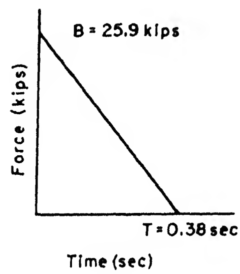




range but not into the plastic range (fig. 7.76, page 208).

a. Design Loading. Since the maximum deflection of this slab is limited to the elasto-plastic region, this maximum deflection occurs in a very short time (table 7.29, page 208). The rapid response means that the design load for all slab elements may be based on the incident overpressure regardless of the direction of motion of the blast wave. Only on the first few feet from the front edge, for the blast wave moving perpendicular to the long axis of the building, is the loading reduced by the vortex action before the slab reaches its maximum deflection (par. 3-08d).

The design load as idealized from the computed loading shown by figure 7.70 is defined by:



$$B = 10 \text{ psi} = \frac{10(144)18}{1000} = 25.9 \text{ kips}$$

$$T = 0.38 \text{ sec}$$

b. Dynamic Design Factors. (Refer to table 6.1.)

astic range:

$K_L = 0.53$, $K_M = 0.41$, $K_{LM} = 0.77$

$R_{lm} = \frac{12M_P}{L}$, $k_1 = \frac{384EI}{L^3}$

$V = 0.36R_n + 0.14P_n$

sto-plastic range:

$K_L = 0.64$, $K_M = 0.50$, $K_{LM} = 0.78$

$R_m = \frac{16M_P}{L}$, $k_{ep} = \frac{384EI}{5L^3}$

$V = 0.39R_n + 0.11P_n$

rage values:

$K_L = 0.5(0.53 + 0.64) = 0.58$

$K_M = 0.5(0.41 + 0.50) = 0.455$

$K_{LM} = 0.5(0.77 + 0.78) = 0.77$

$R_{mf} = \frac{22M_P}{L}$ (fictitious maximum resistance)

$k_E = \frac{264EI}{L^4}$ "elastic" effective spring constant

c. First Trial - Actual Properties.

Assume D.L.F. = 2.0 (experience)

R_{mf} = D.L.F.(B) = 2.0(25.9) = 51.8 kips (par. 6-11)

Assume p = 0.015

$$M_P = pf_{dy}bd^2 \left(1 - \frac{pf_{dy}}{1.7f'_{dc}}\right) \quad \text{(eq 4.16)}$$

$$= 0.015(52)(1)d^2 \left[1 - \frac{0.015(52)}{1.7(3.9)}\right] = 0.688d^2 \text{ kip-ft (d in inches)}$$

$$R_{mf} = \frac{22M_P}{L} = \frac{(22)0.688d^2}{18} = 51.8, \therefore d = 7.85 \text{ in.}$$

Try h = 9-1/2 in., d = 8-1/4 in., p = 0.015, np = 0.15

$$M_P = 0.688d^2 = 0.688(8.25)^2 = 46.9 \text{ kip-ft}$$

$$R_{mf} = \frac{22M_P}{L} = \frac{22(46.9)}{18} = 57.3 \text{ kips}$$

$$I_g = bh^3/12 = (9.5)^3 = 857 \text{ in.}^4$$

$$I_t = bd^3\left[\frac{k^3}{3} + np(1 - k)^2\right] = 12(d)^3\left[\frac{(0.42)^3}{3} + 0.15(1 - 0.42)^2\right]$$

$$= 0.905d^3 = 0.905(8.25)^3 = 508 \text{ in.}^4$$

$$I_a = 0.5(I_g + I_t) = 0.5(857 + 508) = 682 \text{ in.}^4$$

$$k_E = \frac{264EI}{L^3} = \frac{(264)3(10^3)682}{18^3(144)} = 644 \text{ kips/ft}$$

$$\text{Weight} = \left[\frac{9.5(150)}{12} + 6.0\right]\frac{18}{1000} = 2.25 \text{ kips}$$

$$\text{Mass} \quad m = \frac{2.25}{32.2} = 0.07 \text{ kip-sec}^2\text{/ft}$$

d. First Trial - Equivalent Properties.

$$T_n = 2\pi\sqrt{K_{LM}m/k_E} = 6.28\sqrt{0.77(0.07)/644} = 0.0576 \text{ sec}$$

e. First Trial - Available Resistance vs Required Resistance.

$C_T = T/T_n = 0.38/0.0576 = 6.6$

D.L.F. = 1.92 (fig. 5.20)

t_m/T = 0.08 (fig. 5.20)

$t_m = 0.08(0.38) = 0.0304$ sec

Required R_{mf} = D.L.F.(B) = 1.92(25.9) = 49.8 kips < 57.3 kips

The required R_{mf} < available R_{mf}, therefore the selected proportions re satisfactory as a preliminary design.

f. Preliminary Design for Bond Stress.

At $t_m = 0.0304$, $P = \frac{(9.2)144(18)}{1000} = 23.8$ kips (fig. 7.70)

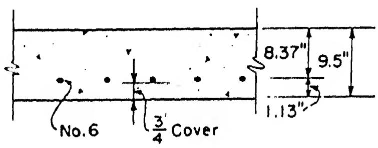
$V = 0.39R_m + 0.11P = 0.39(57.3) + 0.11(23.8) = 22.4 + 2.62 = 25.02$ kips

Allowable $u = 0.15f'_c = 0.15(3000) = 450$ psi (par. 4-09)

$$\Sigma o = \frac{V}{ujd} = \frac{8(25,020)}{450(7)(8.25)} = 7.7 \text{ in.}$$

Try #6 at 3-1/2 in., $A_s = 1.51$ in.^{2}, $\Sigma o = 8.1$ in., $h = 9.5$ in.,

$d = 8.37$, $p = \frac{A_s}{bd} = \frac{1.51}{12(8.37)} = 0.0151$,



$np = 10(0.0151) = 0.151$

g. Determination of Maximum Deflection and Dynamic Reactions by erical Integration.

$$M_p = pf_{dy}bd^2\left(1 - \frac{pf_{dy}}{1.7f'_{dc}}\right) = 0.0153(52)(1)(8.37)^2\left[1 - \frac{(0.0153)(52)}{1.7(3.9)}\right]$$

$$= 49.0 \text{ kip-ft (eq 4.16)}$$

$$I_g = bh^3/12 = (9.5)^3 = 857 \text{ in.}^4$$

$$k = \sqrt{n^2p^2 + 2np} - np = 0.42$$

$$I_t = bd^3\left[\frac{k^3}{3} + np(1 - k)^2\right] = 0.905d^3 = 0.905(8.37)^3 = 530 \text{ in.}^4$$

$$I_a = 0.5(I_g + I_t) = 0.5(857 + 530) = 693 \text{ in.}^4$$

$$\text{Weight} = \left[\frac{9.5(150)}{12} + 6.0\right]\frac{18}{1000} = 2.25 \text{ kips}$$

$$\text{Mass } m = \frac{2.25}{32.2} = 0.07 \text{ kip-sec}^2/\text{ft}$$

tic range:

$$R_{1m} = \frac{12M_p}{L} - \text{weight} = \frac{12(49)}{18} - 2.25 = 32.6 - 2.25 = 30.3 \text{ kips}$$

$$k_1 = \frac{384EI}{L^3} = \frac{384(3)(10)^3 693}{(18)^3(144)} = 950 \text{ kips/ft}$$

$$y_e = \frac{R_{1m}}{k_1} = \frac{30.3}{950} = 0.0319 \text{ ft}$$

Elasto-plastic range:

$$R_m = \frac{16M_P}{L} - \text{weight} = \frac{16(49)}{18} - 2.25 = 43.5 - 2.25 = 41.2 \text{ kips}$$

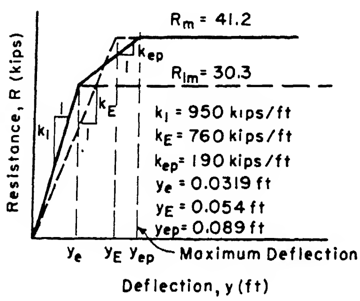


Figure 7.76. Resistance function for 9-1/2-in. slab spanning 18 ft

$$k_{ep} = \frac{384EI}{5L^3} = \frac{1}{5} k_1 = \frac{950}{5} = 190 \text{ kips/ft}$$

$$y_{ep} = y_e + \frac{R_m - R_{1m}}{k_{ep}} = 0.0319 + \frac{41.2 - 30.3}{190}$$

$$= 0.0319 + 0.057 = 0.089 \text{ ft}$$

$$k_E = \frac{307EI}{L^3} = 760 \text{ kips/ft}$$

$$y_E = \frac{R_m}{k_E} = \frac{41.2}{760} = 0.054 \text{ ft}$$

$$T_n = 2\pi\sqrt{K_{LM} m/k_E} = 6.28\sqrt{0.77(0.07)/760} = 0.053 \text{ sec}$$

The basic equation for the numerical integration in table 7.29 is $y_{n+1} = \ddot{y}_n(\Delta t)^2 + 2y_n - y_{n-1}$ (table 5.3) where

Table 7.29. Determination of Maximum Deflection and Dynamic Reactions for Roof Slab

t (sec)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)	V_n (kips)
0	25.9	0	12.9	0.0060	0	3.6
0.005	25.6	5.7	19.9	0.0092	0.0060	5.6
0.010	25.2	20.1	5.1	0.0024	0.0212	10.8
0.015	24.9	31.6	-6.7	-0.0031	0.0388	15.1
0.020	24.5	34.4	-9.9	-0.0045	0.0533	16.1
0.025	24.2	36.3	-12.1	-0.0055	0.0633	16.8
0.030	23.9	37.1	-13.2	-0.0060	0.0678	17.1
0.035	23.5	35.7	-12.2	-0.0057	0.0663	16.1
0.040	23.2	28.8			0.0591	13.6
0.045	22.8					11.4
0.050	22.5					11.2

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(\Delta t)^2}{K_{LM}(m)}$$

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(0.005)^2}{0.77(0.07)} = 4.638(10^{-4})(P_n - R_n) \text{ ft, elastic range}$$

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(0.005)^2}{0.78(0.07)} = 4.578(10^{-4})(P_n - R_n) \text{ ft, elasto-plastic range}$$

The time interval Δt = 0.005 sec is approximately $T_n/10$ = 0.0053 ar. 5-08). The dynamic reaction equations are listed in paragraph 7-49b. e P_n values for the second column are obtained from figure 7.70, multiying by 144(18)/1000 = 2.59.

The maximum deflection $(y_n)_{max}$ computed in table 7.29 is 0.0678 ft ch is less than the allowable y_m of 0.089 ft.

h. Shear Strength and Bond Stress.

V_{max} = 17.0 kips (table 7.29)

For no shear reinforcement

Allowable $v_y = 0.04f'_c + 5000p$ (eq 4.24)

$v_y = 0.04(3000) + 5000(0.0151) = 120 + 76 = 196$ psi

$$v = \frac{8V}{7bd} = \frac{8(17{,}000)}{7(12)(8.37)} = 193 \text{ psi; OK}$$

No shear reinforcement required

$$u = \frac{8V}{7\Sigma od} = \frac{8(17{,}000)}{7(8.1)8.37} = 287 \text{ psi}$$

Allowable $u = 0.15f'_c = 0.15(3000) = 450$ psi; OK

i. Summary.

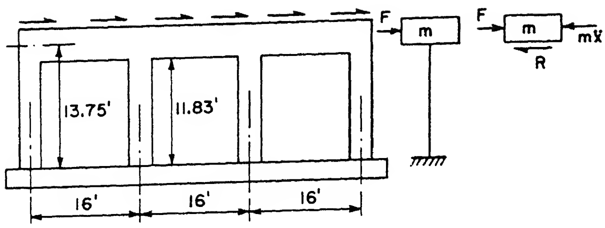
9-1/2-in. slab

p = 0.0151

Σo = 8.1 in.

No shear reinforcement required

0 PRELIMINARY DESIGN OF COLUMN. A single-story frame subject to lateral d behaves essentially as a single-degree-of-freedom system. In determin- the requirement for the columns which are the springs of the system it not necessary to use the equivalent system technique otherwise used in s manual for designing structural elements.



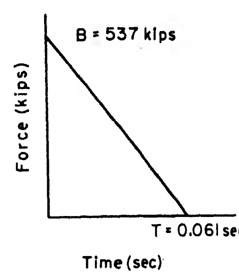
Combining the principles of paragraph 6-11 and the equations from paragraph 7-06 results in a procedure for preliminary design. In this preliminary design the girders are assumed to be infinitely rigid to simplify the analysis.

The column height from the girder centerline to the top of the footing is 13.75 ft. This dimension is used to determine the spring constant. The maximum resistance computation is based on the clear column height, 11.83 ft.

a. Design Loading. In the preliminary design it is an unnecessary refinement to use the dynamic reactions from the wall slabs. The design load is idealized from the net lateral overpressure curve. The height of wall considered to load the frame is equal to half the clear height of the wall plus the thickness of the roof slab

$$h' = \frac{14.75}{2} + 0.79 = 8.16 \text{ ft}$$

The design load as idealized from the computed loading shown by figure 7.73 is defined by:



$$B = 25.3 \text{ psi} = \frac{25.3(144)18(8.16)}{1000} = 537 \text{ kips}$$

$$T = 0.061 \text{ sec}$$

b. Mass Computation.

$$\text{Roof slab} \left[\frac{9.5(150)}{12} + 6.0\right] \frac{18(54)}{1000} = 122 \text{ kips}$$

$$\text{Girder (assumed)} \ \frac{18(32)50.5(150)}{12(12)1000} = 30 \text{ kips}$$

$$\text{4 columns (assumed)} \ \frac{18(30)(11.83)150(4)}{12(12)(1000)} = 26.6 \text{ kips}$$

$$\text{2 wall slabs} \ \frac{12.5(14.75)150(18)2}{12(1000)} = 83.0 \text{ kips}$$

Mass of single-degree-of-freedom system = total roof slab + total girder + $\frac{1}{3}$ columns + $\frac{1}{3}$ walls = $\frac{122 + 30 + 0.33(26.6 + 83.0)}{32.2}$ = 5.85 kip-sec^2/ft

c. First Trial - Actual Properties.

Assume D.L.F. = 1.2 (experience)

R_m = D.L.F.(B) = 1.2(537) = 645 kips (par. 6-11)

Required $M_D = R_m h/2n = 645(11.83)/2(4) = 955$ kip-ft

Estimated average roof pressure = 6.4 psi (fig. 7.74)

Average blast load per column $= \dfrac{54(18)6.4(144)}{1000(4)} = 224$ kips

Dead load per column $= \dfrac{1}{4}(122 + 30) = 38$ kips

Average column design load $= P_D = 224 + 38 = 262$ kips

$$M_D = A_s f_{dy} d' + P_D \left(0.5t - \frac{P_D}{1.7bf'_{dc}}\right) \quad \text{(eq 4.32)}$$

Let $p = p' = 0.015$, $d' = (t - 7.5)$ in., $d'' = 3.75$ in., $b = 18$,

$A_s = pbt$

Substituting into previous M_D equation and solving for t gives

$$955(12) = 0.015(18)t(52)(t - 7.5) + 262\left[0.5t - \frac{262}{1.7(18)3.9}\right]$$

$14.0t^2 + 25.0t - 12{,}037 = 0$, solving $t = 28.4$ in.

Try section 18 in. by 28 in., $d = 28 - 3.75 = 24.25$ in.

Assume $d''/d = 3.75/24.25 = 0.155$,

$m = np + (n - 1)p' = 0.15 + 9(0.015) = 0.285$

$q = np + (n - 1)p'\, d''/d = 0.15 + 9(0.015)0.155 = 0.171$

$k = 0.37$ (table 11 RCDH of ACI)

$$I_g = \frac{bt^3}{12} = \frac{18(28)^3}{12} = 32{,}900 \text{ in.}^4$$

$$I_t = bd^3\left[\frac{k^3}{3} + (n - 1)p'\left\{k^2 - 2k(d''/d) + (d''/d)^2\right\} + np(1 - k)^2\right]$$

$$= 18(24.25)^3\left[[(0.37)^3/3] + 9(0.015)\left\{(0.37)^2 - 2(0.37)(0.155) + (0.155)^2\right\} + 0.15(1 - 0.37)^2\right] = 258{,}000\left[0.0169 + 0.135(0.137 - 0.115 + 0.024) + 0.0595\right] = 258{,}000(0.0826) = 21{,}300 \text{ in.}^4$$

$I_a = 0.5(I_g + I_t) = 0.5(32{,}900 + 21{,}300) = 27{,}100$ in.^{4}

d. First Trial - Determination of D.L.F.

$$k = \frac{12EIn}{h^3} = \frac{12(3)10^3(27{,}100)4}{(13.75)^3 144} = 10{,}450 \text{ kips/ft (eq 7.10)}$$

$x_e = R_m/k = 645/10{,}450 = 0.062$ ft

$$T_n = 2\pi\sqrt{m/k} = 6.28\sqrt{5.85/10{,}450} = 0.148 \text{ sec}$$

$$T/T_n = 0.061/0.148 = 0.41$$

$$\text{D.L.F.} = 1.06,\ t_m/T = 0.94 \text{ (fig. 5.20)}$$

$$t_m = 0.94(0.061) = 0.0575 \text{ sec}$$

The original idealized load-time curve should be revised to obtain a closer approximation to the total impulse up to time t_m. The impulse up to $t = 0.060$ sec in figure 7.73 is

$$H = 0.898 \text{ psi-sec (obtained by graphical integration)}$$

$$T = 2H/B = 2(0.898)/25.3 = 0.071 \text{ sec}$$

$$T/T_n = 0.071/0.148 = 0.48$$

$$\text{D.L.F.} = 1.15,\ t_m/T = 0.85 \text{ (fig. 5.20)}$$

The revised idealized load is satisfactory because $t_m = 0.85(0.071) = 0.060$ sec

$$\text{Required moment} = \frac{(B)\text{D.L.F.}(h)}{2n} = \frac{537(1.15)11.83}{2(4)} = 915 \text{ kip-ft}$$

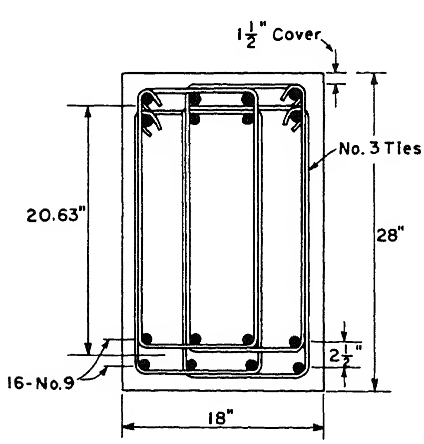
Required moment is less than available moment. Let us investigate possible reduction in size.

e. <u>Second Trial - Actual Properties.</u>

$$M_D = A_s f_{dy} d' + P_D\left(0.5t - \frac{P_D}{1.7bf'_{dc}}\right) \quad \text{(eq 4.32)}$$

$$(915)12 = 0.015(18)t(52)(t - 7.5) + 262\left[0.5t - \frac{262}{1.7(18)3.9}\right]$$

$$14.0t^2 + 25.0t - 11{,}547 = 0, \text{ solving } t = 27.8 \text{ in.}$$



The first trial proportions are satisfactory. An actual column section is selected to establish the column plastic bending moment for use in the girder design that follows in paragraph 7-51.

$$d'' = 3.687,\ d = 24.32,\ d' = 20.63$$

$$8\ \#9,\ A_s = A'_s = 8.0 \text{ in.}^2$$

$$p = p' = 8.0/18(28) = 0.0159$$

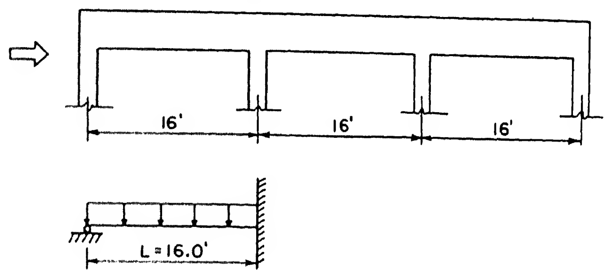
$$M_D = 8.0(52)(20.63) + 262\left[14 - \frac{262}{1.7(18)3.9}\right] \quad \text{(eq 4.32)}$$

$$= 8600 + 3070 = 11{,}670 \text{ kip-in.} = 970 \text{ kip-ft}$$

The previously computed value of I will be satisfactory for the computation in paragraph 7-51.

7-51 DESIGN OF ROOF GIRDER. The design of the girder in this example is performed in the same manner as the girder in paragraph 7-43 since both girders are designed for elastic action. Reference should be made to paragraph 7-11 for a detailed discussion of the design of roof girders.

The front girder span is critical. Previous consideration of the variation of local roof overpressure with location along the span from front to back of the building has shown that for the front girder the incident overpressure may be used as the local roof overpressure because the maximum response of the girder generally occurs before the vortex action has an opportunity to cause the local roof loading to vary strongly from the incident overpressure. This can be seen from a comparison of the times for maximum displacement of the girders in tables 7.9, 7.21, and 7.26 with the local overpressure data in paragraph 7-23.



A fixed-pinned tee beam is considered. Reference is made to paragraph 6-20 and table 6.4 for design constants.

a. Load Determination. The girder loading is obtained in figure 7.77 from the numerical integration for the roof slab (par. 7-49d, table 7.29).

After $t = 0.045$ sec the load on the girder from the slab is assumed to be equal to one-half the load on the slab. The time required for the shock front to traverse the girder is considered in establishing the load-time curve by plotting the dynamic slab reactions at the ends of the girder and averaging over the span. The time required for the shock wave to travel the length of the girder is

$$t_{lag} = \frac{16}{1403} = 0.0114 \text{ sec}$$

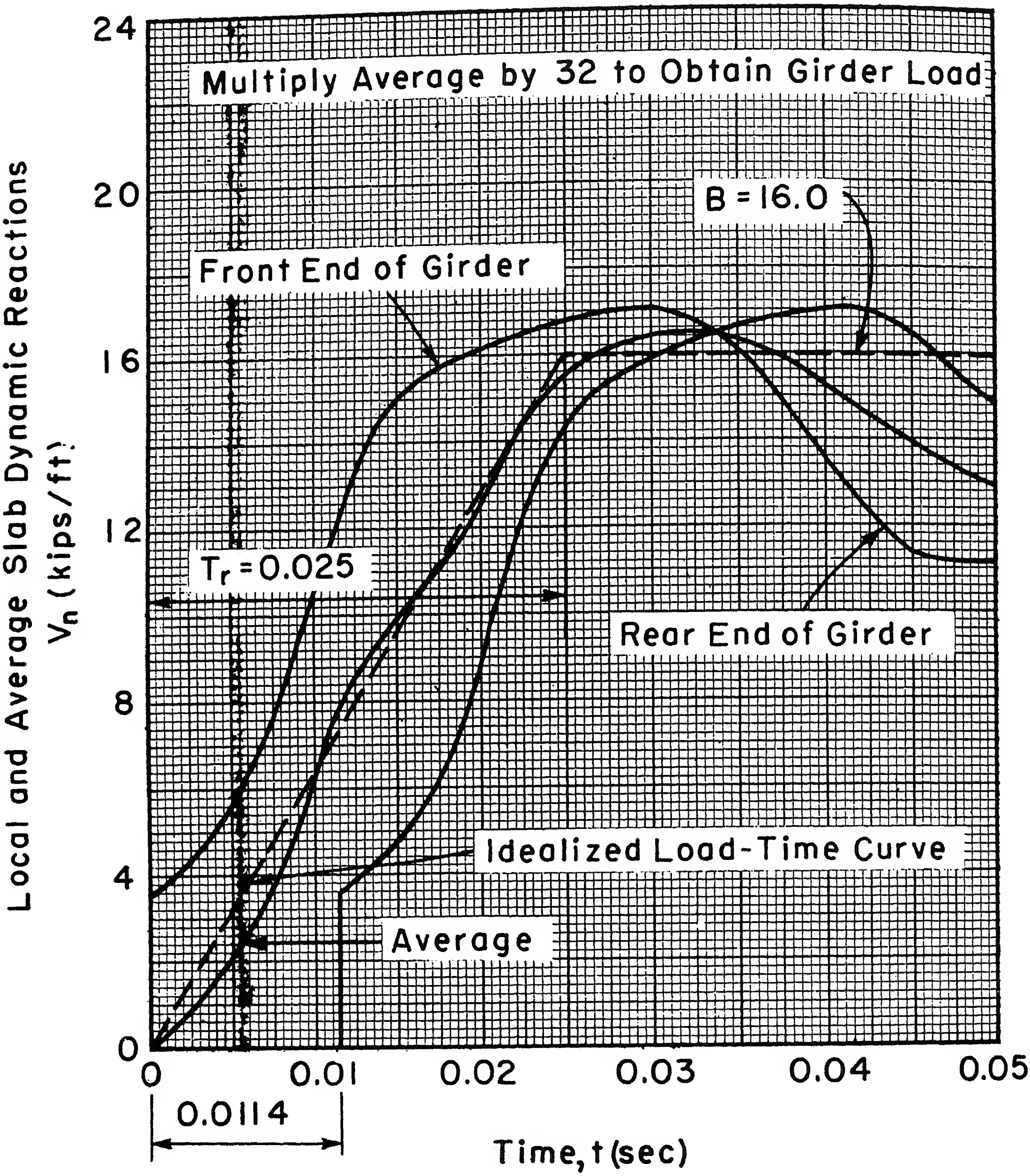
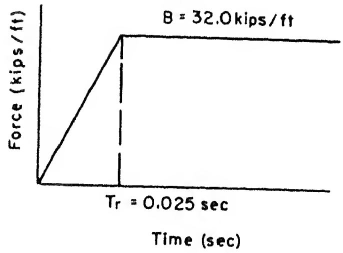


Figure 7.77. Local and average slab dynamic reactions for incident overpressure, blast wave parallel to girder

The idealized load-time curve imposed on the roof girder by a slab on one side is indicated by the dashed line in fig. 7.77. The design load on the girder is defined by



$B = 2(16.0) = 32.0$ kips/ft

$T_r = 0.025$ sec

b. Elastic Range Dynamic Design Factors. (Refer to table 6.4.)

$K_L = 0.58$, $K_M = 0.45$, $K_{LM} = 0.78$

$k = f_3 EI/L^3$ (fig. 6.29)

$R_m = f_1 M_{Ps}/L$ (fig. 6.27)

$M_{pos} = f_2 M_n$ (fig. 6.28), $M_n = R_1 L/f_1$ (fig. 6.27)

$V_1 = 0.26R + 0.12P$

$V_2 = 0.43R + 0.19P$

c. Mass Computation.

$$\text{Slab and roofing} = \left[\frac{9.5(150)}{12} + 6.0\right]\frac{18(54)}{1000(3)} = 40.5 \text{ kips}$$

$$\text{Girder (estimate)} = \frac{18(36)150(16)}{12(12)1000} = 10.8 \text{ kips}$$

$$\text{Total mass} = \frac{40.5 + 10.8}{32.2} = 1.59 \text{ kip-sec}^2/\text{ft}$$

d. First Trial - Actual Properties.

Estimate tee beam action for first trial, $f_1 = 9$, $f_2 = 0.67$, $f_3 = 240$

Assume D.L.F.(B) = 1.05 (experience)

R_m = D.L.F.(B) = 1.05(32.0)16 = 540 kips

Moment in girder at interior support (fixed support) due to static loads

$$M = \frac{WL}{8} = \frac{(40.5 + 10.8)16}{8} = 103 \text{ kip-ft}$$

Moment in girder at midspan due to static loads

$$M = \frac{9WL}{128} = \frac{9(51.3)16}{1.28} = 58 \text{ kip-ft}$$

Support moment from column

$\frac{2}{3} M_D = \frac{2}{3} (970) = 646$ kip-ft (par. 7-11 and 7-50)

Midspan moment from column = 0

Support moment required for vertical blast loads

$$M = \frac{R_m L}{9} = \frac{540(16)}{9} = 960 \text{ kip-ft}$$

Midspan moment due to vertical blast loads

$M_{pos} = f_2 M_n = 0.67(960) = 640$ kip-ft

Total support moment = 960 + 646 + 103 = 1709 kip-ft

Total midspan moment = 640 + 0 + 58 = 698 kip-ft

Ratio of midspan tension reinforcing to interior support reinforcing steel = 698/1709 = 0.41

$$M_p = pf_{dy}bd^2\left(1 - \frac{pf_{dy}}{1.7f'_{dc}}\right) = 0.015(52)bd^2\left[1 - \frac{0.015(52)}{1.7(3.9)}\right]$$

$M_p = 0.688bd^2 = 1709(12) = 20{,}500$ kip-in.

Try b = 20 in.

$d = \sqrt{20{,}500/13.76} = 38.6$ in.

Try d = 38.5, h = 42 in.

Rectangular section at the support

$I_g = 20(42)^3/12 = 123{,}500 \text{ in.}^4$

$np = 10(12)/20(38.5) = 0.156$

$np' = 10(6)/20(38.5) = 0.078$

$m = np + (n - 1)p' = 0.156 + \frac{9}{10}(0.078) = 0.226$

$q = np + (n - 1)p' \frac{d'}{d} = 0.156 + \frac{9}{10}(0.078)\frac{2.25}{38.5} = 0.160$

k = 0.39 (table 11, RCDH of ACI)

kd = 0.39(38.5) = 15.0 in.

$$I_t = \frac{20(15)^3}{3} + 12(10)(23.5)^2 + 9(6)(12.75)^2$$

$$= 22{,}500 + 66{,}400 + 8{,}800 = 97{,}700 \text{ in.}^4$$

$I_1 = (I_g + I_t)0.5 = 110{,}600 \text{ in.}^4$

Tee section at midspan

Tension reinforcement at midspan [illegible] ([illegible]) [illegible] 4.[illegible], use 5 bars

$$\bar{y} = \frac{\frac{-96(9.5)^2}{2} + \frac{20(32.5)^2}{2}}{96(9.5) + 20(32.5)} = \frac{-4330 + [illegible]}{912 + 650} = \frac{[illegible]70}{156[illegible]} = 4.0 \text{ in.}$$

$$I_g = \frac{96(9.5)^3}{3} + \frac{20(32.5)^3}{3} - 1562(4.0)^2 = 27,400 + 229,000 - 25,000$$

$$= 231,400 \text{ in.}^4$$

$np = 10(5)/39.13(96) = 0.0133$

$np' = 10(3)/39.13(96) = 0.00[illegible]$

$m = np + (n - 1)p' = 0.0133 + \frac{9}{10}(0.[illegible]) = 0.0[illegible]$

$q = np + (n - 1)p' \frac{d'}{d} = 0.0133 + \frac{9}{10}(0.[illegible]) \frac{[illegible]}{39.75} = 0.0137$

k = 0.145 (table [illegible], RCDH of ACI)

$kd = 0.145(39.75) = 5.76$ in.

$$I_t = \frac{96(5.76)^3}{3} + 9(4)(5.76 - 2.25)^2 + 10(5)(39.75 - 5.76)^2$$

$I_t = 6150 + 443 + [illegible] = [illegible]$ in.^{4}

$I_2 = 0.5(I_g + I_t) = [illegible]$ in.^{4}

$I_1/I_2 = 110,600/[illegible] = [illegible]$

$f_1 = 9.0$ (fig. [illegible])

$f_2 = 0.68$ (fig. [illegible])

$f_3 = 246$ (fig. [illegible])

e. First Trial - Determination of D.L.F.

$$k_1 = \frac{f_1 E I_1}{L^3} = \frac{246(3)([illegible])^3([illegible])}{([illegible])^3 144} = 138,000 \text{ kips/ft}$$

$T_n = 2\pi\sqrt{K_{LM} m/k_1} = 6.28\sqrt{0.7[illegible](1.59)/138,000} = 0.0189$ sec

$T_r/T_n = 0.025/0.0189 = 1.3[illegible]$

D.L.F. = 1.2 (fig. 5.21)

$t_m/T_r \approx 1.25$ (fig. 5.21); OK (see fig. 7-77)

Required $R = 1.2(32.0)(16) = 615$ kips

At the interior support section

$$M_p = \frac{0.0156(52)20(38.5)^2}{12}\left[1 - \frac{0.0156(52)}{1.7(3.9)}\right] = 1760 \text{ kip-ft}$$

The moment available at the support to resist the effects of vertical blast loads is

$M = 1760 - 646 - 103 = 1011$ kip-ft

The available resistance is then

$R_m = f_1M/L = 9.0(1011)/16 = 569.0$ kips

This girder is inadequate, $569 < 615$

Increase reinforcing at support to 13 bars and continue with same girder. The moments of inertia will not change appreciably (par. 7-43).

f. Preliminary Design for Bond Stress.

Estimated $V_{max} = 0.62R_m = 0.62(615) = 381$ kips

Allowable $u = 0.15f'_c = 0.15(3000) = 450$ psi (par. 4-09)

$$\Sigma o = \frac{V}{ujd} = \frac{8(381,000)}{450(7)(38.5)} = 25.0 \text{ in.}$$

A_s = 13 #9 bars in two rows, $A_s = 13$ in.^{2}

$$\Sigma o = 46.0 \text{ in.}, \; p = \frac{A_s}{bd} = \frac{13}{20(38.5)} = 0.0169$$

g. Determination of Maximum Deflection and Dynamic Reactions by Numerical Integration. Since the trial size is to be used for the numerical integration with only minor modification, reference is made to the previous computations for pertinent data.

$$\text{Revised girder weight} = \frac{20(32.5)16(150)}{12(12)1000} = 10.8 \text{ kips (same as estimate)}$$

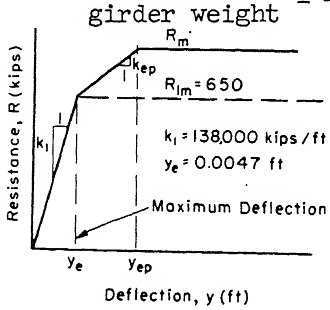


Figure 7.78. Resistance function for girder spanning 16 ft

Total mass = 1.59 kip-sec^2/ft

$$M = 1760\,\frac{13}{12} - 646 - 103 = 1156 \text{ kip-ft}$$

$$R_m = \frac{f_1M}{L} = \frac{9.0(1156)}{16} = 650 \text{ kips}$$

$k_1 = 138,000$ kips/ft

$$y_e = \frac{R_{1m}}{k_1} = \frac{650}{138,000} = 0.0047 \text{ ft}$$

Table 7.30. Determination of Maximum Deflection and Dynamic Reactions for Girder

t (sec)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)	V_{2n} (kips)	V_{1n} (kips)
0	0	0	3	0.000010	0		
0.002	19	1	18	0.000058	0.000010		
0.004	51	11	40	0.000129	0.000078		
0.006	90	38	52	0.000167	0.000275		
0.008	141	88	53	0.000171	0.000639		
0.010	202	162	40	0.000129	0.001174		
0.012	262	254	8	0.000026	0.001838		
0.014	301	349	-48	-0.000155	0.002526		
0.016	333	423	-90	-0.000290	0.003063		
0.018	365	456	-91	-0.000293	0.003308		
0.020	403	450	-47	-0.000151	0.003260		
0.022	454	422	32	0.000103	0.003061		
0.024	486	409	77	0.000248	0.002965		
0.026	506	430	76	0.000245	0.003117		
0.028	518	485	33	0.000106	0.003514		
0.030	525	554	-29	-0.000093	0.004017		
0.032	528	611	-83	-0.000267	0.004427		
0.034	525	631	-106	-0.000341	0.004570	371	227
0.036	518	603	-85	-0.000274	0.004372		
0.038	506				0.003900		

The basic equation for the numerical integration in table 7.30 is $y_{n+1} = \ddot{y}_n(\Delta t)^2 + 2y_n - y_{n-1}$ (table 5.3)

$$\ddot{y}_n(\Delta t)^2 = \frac{(P_n - R_n)(\Delta t)^2}{K_{LM}m} = \frac{(P_n - R_n)(0.002)^2}{0.78(1.59)} = 3.22(10^{-6})(P_n - R_n) \text{ ft}$$

In table 7.30 the time interval $\Delta t = 0.002$ sec is approximately $T_n/10 = 0.00189$ sec (par. 5-08). The dynamic reaction equations are listed

in paragraph 7-51b. The P_n values for the second column are obtained from figure 7.77 multiplying by 2(16) = 32 to account for the 16-ft-span length of the girder and the two slabs loading the girder. The $(y_n)_{max}$ value $= 0.0044$ ft $\approx y_e$. The design is satisfactory for this consideration.

h. Shear and Bond Strength. For interior support end of girder (fixed end of idealized girder)

$V_{max} = 371$ kips (table 7.30)

For no shear reinforcement allowable $v_y = 0.04f'_c + 5000p$ (eq 4.24)

$v_y = 0.04(3000) + 5000(0.0156) = 120 + 78 = 198$ psi

$$v = \frac{8V}{7bd} = \frac{8(371{,}000)}{7(20)38.55} = 550 \text{ psi}$$

Shear reinforcement required for 550 - 198 = 352 psi

Contribution of shear reinforcement to allowable shear stress = rf_y

$$r \doteq \frac{352}{40{,}000} = 0.0088$$

Try 4 #4 bars, $A_s = 0.80$ in.^{2}

$$r = \frac{A_s}{bs} = \frac{0.80}{20s} = 0.0088; \therefore s = 4.55 \text{ in., try } s = 4 \text{ in.}$$

For exterior support end of girder (pinned end of idealized girder)

$V_{max} = 227$ kips (table 7.30)

$v_y = 0.04(3000) + 5000(0.00133) = 120 + 6.6 = 127$ psi

$$v = \frac{8V}{7bd} = \frac{8(227{,}000)}{7(20)39.75} = 326 \text{ psi}$$

Shear reinforcement required for 326 - 127 = 199 psi

$$r = \frac{199}{40{,}000} = 0.00498$$

Try 4 #4 bars, $A_s = 0.80$ in.^{2}

$$r = \frac{A_s}{bs} = \frac{0.80}{20(s)} = 0.00498; \therefore s = 8.0 \text{ in. try } s = 8 \text{ in.}$$

$$u = \frac{8V}{7\Sigma o d} = \frac{8(371{,}000)}{7(46.0)38.55} = 238 \text{ psi}$$

Allowable $u = 0.15f'_c = 0.15(3000) = 450$ psi; OK

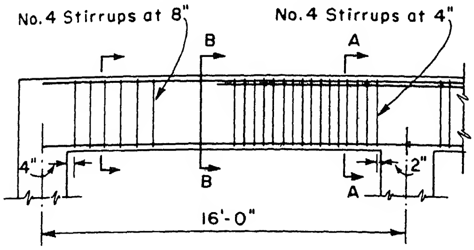
i. Summary.

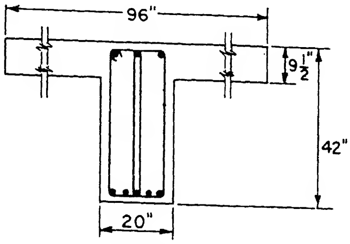
20-in. by 42-in. tee beam

Maximum tension reinforcement = 13 #9 bars

$p = 0.0156$

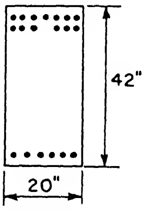
$\Sigma o = 46$ in.





Effective Section at B-B

Note: 2" Protection for All Steel



Effective Section at A-A

Shear reinforcement at interior end of girder, 4 #4 at 4 in.

Shear reinforcement at exterior end of girder, 4 #4 at 8 in.

7-52 FINAL DESIGN OF COLUMN. This column design for elastic behavior was begun in paragraph 7-50. The calculations which follow illustrate the steps which are needed to determine the adequacy of the preliminary design. The primary computation is a numerical integration in which is determined the time variation of the lateral displacement of the top of the columns.

In the preliminary design idealized loads and resistances are used in order to simplify the computations. In the final design, however, these idealizations are no longer used. This results in consideration of the following factors which have been neglected: (1) the variation of plastic hinge moment with direct stress, (2) the variation of column resistance with lateral deflection, (3) the effect of girder flexibility on the stiffness of the frame, and (4) the difference between the two design loads; one determined from the product of the front face overpressure and the wall area and the other determined from the wall slab dynamic reactions.

a. Mass Computation.

Roof slab = 122 kips (par. 7-50b)

$$\text{Girder stem} = \frac{20(35.5)50.5(150)}{12(12)1000} = 37.3 \text{ kips}$$

$$\text{Columns} = \frac{18(28)11.75(150)4}{12(12)1000} = 24.6 \text{ kips}$$

Walls = 83.0 kips (par. 7-50b)

Mass of single-degree-of-freedom system = total roof slab +

$$\text{total girder} + \frac{1}{3}\ (\text{columns} + \text{walls}) = \frac{122 + 37.3 + 0.33(24.6 + 83.0)}{32.2}$$

$= 6.05$ kip-sec^2/ft

b. Column Properties. (See par. 7-50e.)

$b = 18$ in., $t = 28$ in., $d = 24.32$ in., $d' = 20.63$ in.

$d'' = 3.687$ in., $A_s = A'_s = 8.0$ in.^{2}, $p = p' = \dfrac{8}{18(24.32)} = 0.0183$

$$M_D = A_s f_{dy} d' + P_D\left(0.5t - \frac{P_D}{1.7bf'_{dc}}\right) \quad \text{(eq 4.32)}$$

$$= \frac{8.0(52)20.63}{12} + \frac{P_D}{12}\left[14 - \frac{P_D}{1.7(18)3.9}\right]$$

$$M_D = 716 + 1.16P_D - 0.0007P_D^{\ 2}$$

c. Effect of Girder Flexibility. Consideration of the relative flexibility of the girders and columns generally results in a value of the frame elastic spring constant k which is less than the value obtained for the assumption of infinitely stiff girders (par. 7-08) in the preliminary design. To obtain this revised value a simple sidesway analysis of the frame is made. From the sidesway analysis, k is the magnitude of the lateral load required to cause unit displacement.

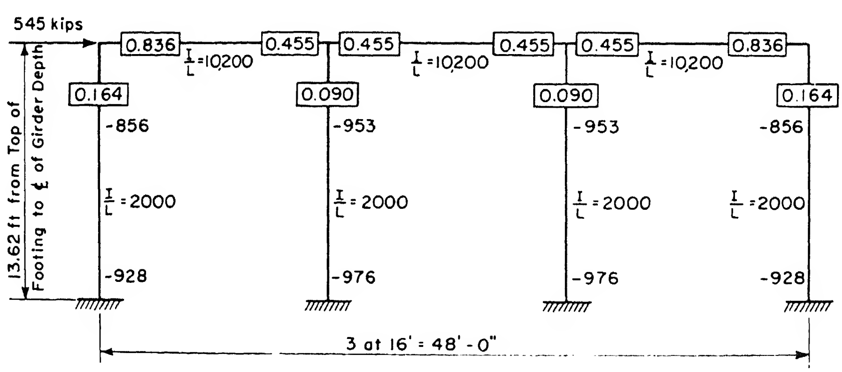


Figure 7.79. Sidesway analysis by moment distribution

The elastic sidesway analysis (the results of which are shown in figure 7.79) is performed for initial column moments of 1000 kip-ft at top and bottom of each column. This is equivalent to a lateral displacement of the top of the column.

$$x = \frac{(F.E.M.)h^2}{6EI} = \frac{1000(13.62)^2 144}{6(3)10^3(27,250)} = 0.0545 \text{ ft}$$

From figure 7.79

$$R = \frac{\Sigma M}{h} = \frac{2(856 + 928 + 953 + 976)}{13.62} = 545 \text{ kips}$$

$$k = \frac{R}{x} = \frac{545}{0.0545} = 10,000 \text{ kips/ft}$$

d. Loading. The F_n column of the numerical integration analysis (table 7.31) is obtained from figure 7.80. The first part of figure 7.80

Table 7.31. Determination of Column Adequacy

t (sec)	P_n (kips)	$(P_{av})_n$ (kips)	$(M_D)_n$ (kip-ft)	$(R_m)_n$ (kips)	F_n (kips)	R_n (kips)	$F_n - R_n$ (kips)	$\ddot{x}_n(\Delta t)^2$ (ft)	x_n (ft)
0	159	40	761	517	200	0	100.0	0.00165	0
0.01	467	117	843	573	475	16.5	458.5	0.00757	0.00165
0.02	775	194	915	622	475	108.7	366.3	0.00604	0.01087
0.03	1083	271	979	666	275	261.3	13.7	0.00023	0.02613
0.04	1358	340	1029	699	230	416.2	-186.2	-0.00307	0.04162
0.05	1308	327	1020	694	185	540.4	-355.4	-0.00586	0.05404
0.06	1259	315	1012	689	144	606.0	-462.0	-0.00762	0.06060*
0.07									0.05954

* $(x_n)_{max}$ = 0.061 ft

is based on the V_{1n} dynamic reaction column of the wall slab analysis (table 7.28). The dynamic reactions for a one-foot width of wall are multiplied by 18, the width of one frame bay. The portion of the curve after t = 0.025 sec is based on the net lateral overpressure curve (fig. 7.73). Here the values of F_n are obtained from $\overline{P}_{net}$ in psi by using

$$F_n = \frac{144(18)8.16}{1000} \overline{P}_{net} = 21.2\overline{P}_{net} \text{ kips}$$

The dimension 8.16 is equal to one-half the front wall clear span plus the roof slab thickness $\frac{(14.75)}{2} + 0.79 = 8.16$ ft.

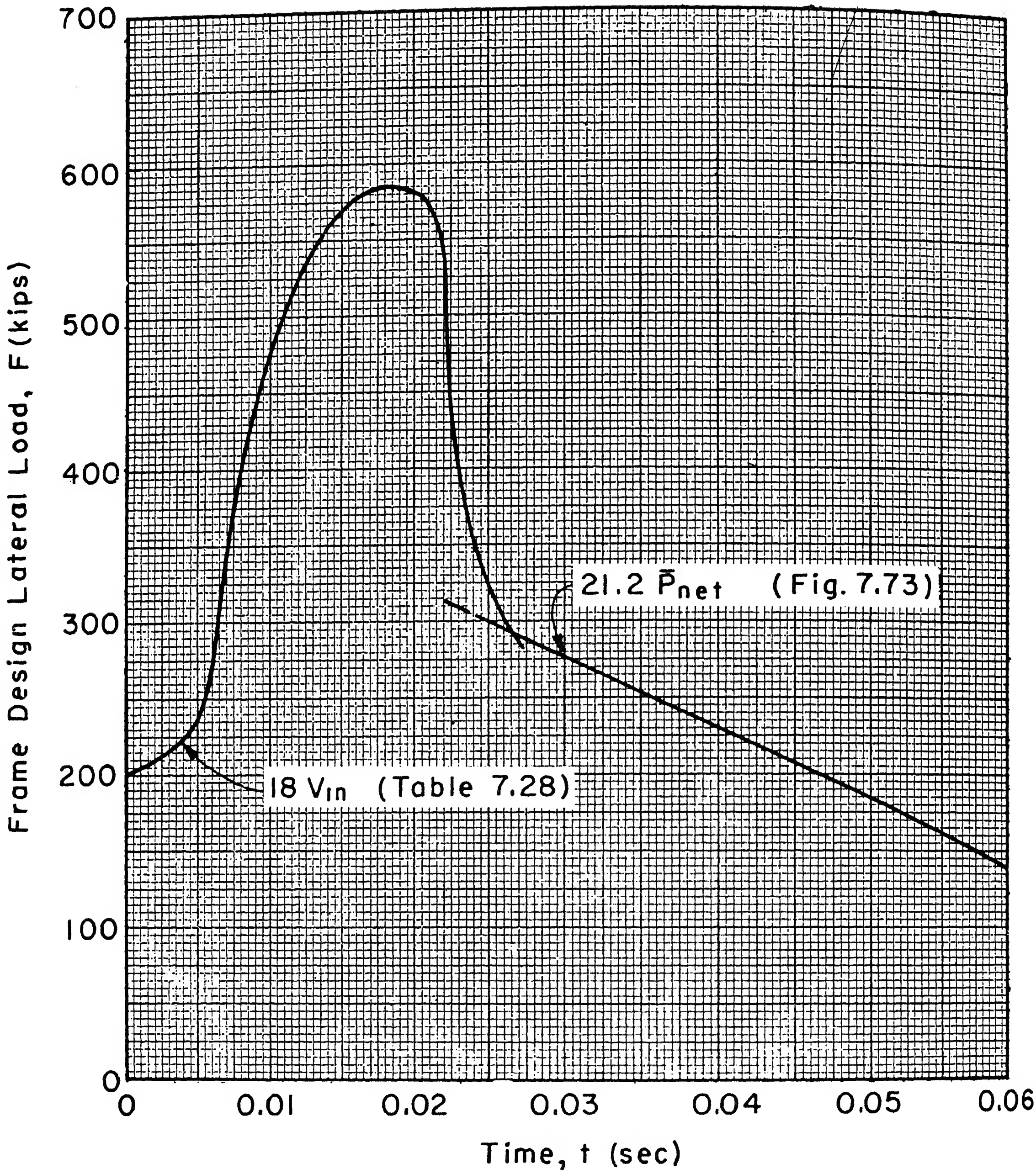


Figure 7.80. Frame design lateral load

e. <u>Numerical Integration Computation to Determine Column Adequacy.</u> The total vertical load P_n in the second column is obtained by multiplying the average roof overpressure (fig. 7.74) by [54(18)144]/1000 = 140 and adding the dead weight of the roof system (159.3 kips). The $(P_{av})_n$ values are

the average axial column loads and are obtained by dividing the total vertical load by the number of columns, 4. The $(P_{av})_n$ column is used as P_D in the formula of paragraph 7-52b to obtain the value of $(M_D)_n$. The value of $(M_D)_n$ is used in turn to obtain the maximum resistance at any time from the relation

$$(R_m)_n = \frac{2n}{h_c}(M_D)_n = \frac{2(4)(M_D)_n}{11.75} = 0.68(M_D)_n$$

R_n is equal to $kx_n = 10{,}000x_n$ in the elastic range.

The basic equation for the numerical integration analysis in table 7.31 is

$$x_{n+1} = \ddot{x}_n(\Delta t)^2 + 2x_n - x_{n-1} \quad \text{(table 5.3)}$$

where

$$\ddot{x}_n(\Delta t)^2 = \frac{(F_n - R_n)}{m}(\Delta t)^2 = \frac{(F_n - R_n)}{6.05}(0.01)^2 = 0.165(10^{-4})(F_n - R_n) \text{ ft}$$

The time interval $\Delta t = 0.01$ sec used in table 7.31 is based on the natural period, $T_n = 2\pi\sqrt{m/k} = 6.28\sqrt{6.05/10{,}000} = 0.1545$ sec

$\Delta t = 0.01 < T_n/10 = 0.01545$ sec (par. 5-08)

In table 7.31 the design is elastic because $R_n < (R_m)_n$, i.e., $606 < 689$.

f. Shear and Bond Stress.

$$V_{max} = \frac{606}{4} = 151.5 \text{ kips (table 7.31)}$$

For no shear reinforcement,

Allowable $v_y = 0.04f'_c + 5000p$ (eq 4.24)

$v_y = 0.04(3000) + 5000(0.0183) = 120 + 90 = 210$ psi

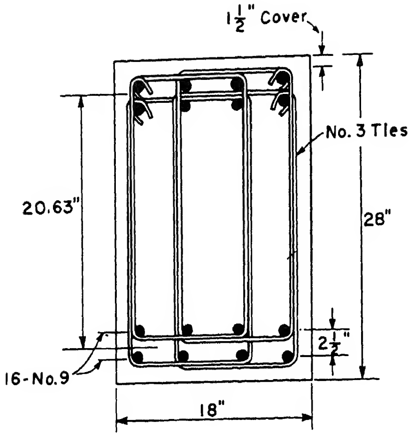
$$v = \frac{8V}{7bd} = \frac{8(151{,}500)}{7(18)(24.32)} = 395 \text{ psi}$$

Shear reinforcement required for $395 - 210 = 185$ psi

3 #3 column ties, $A_s = 6(0.11) = 0.66$ in.^{2}

$$r = \frac{185}{40{,}000} = 0.00462$$

$$r = \frac{A_s}{bs} = 0.00462 = \frac{0.66}{18(s)}\ ; \therefore \quad s = 7.95 \text{ in., use } \quad s = 8 \text{ in.}$$



$\Sigma o = 32.0$ in.

$$u = \frac{8(151{,}500)}{7(32.0)(24.32)} = 222 \text{ psi}$$

Allowable $u = 0.15f'_c = 0.15(3000)$

$= 450$ psi; OK

g. Summary.

18 in. by 28 in. tied column

$A_s = A'_s = 8.0$ in.^{2}

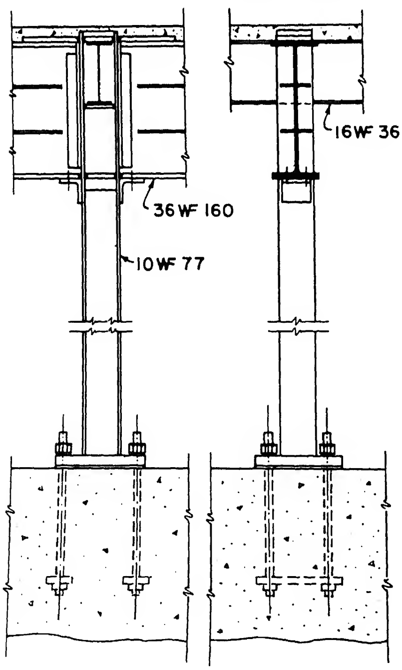
3 #3 ties at 8 in.

$\Sigma o = 32.0$ in.

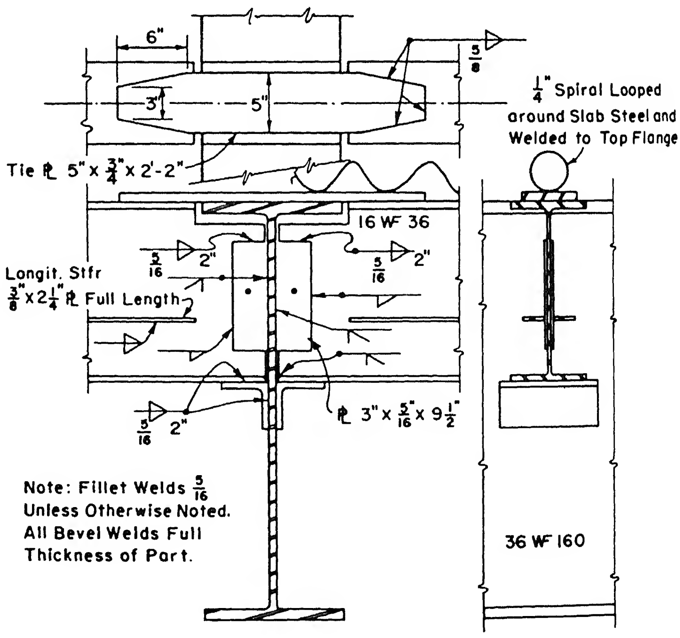
DESIGN DETAILS

7-53 STRUCTURAL STEEL DETAILS. To illustrate the type of design details that might be necessary, typical details for purlin to girder, girder to column, and column to foundation connections are presented.

Purlin to Girder Connection. The heavy end reactions and moments make necessary both seat angles and web plates. The web plates stiffen the purlin web near the connection where the longitudinal stiffeners must



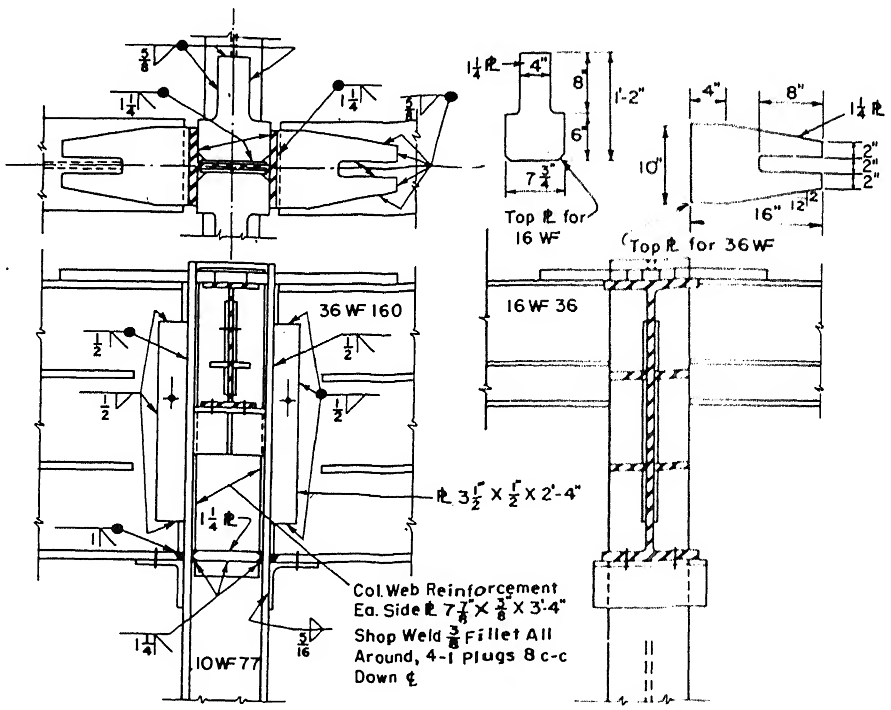
Middle Column



Purlin to Girder Connection

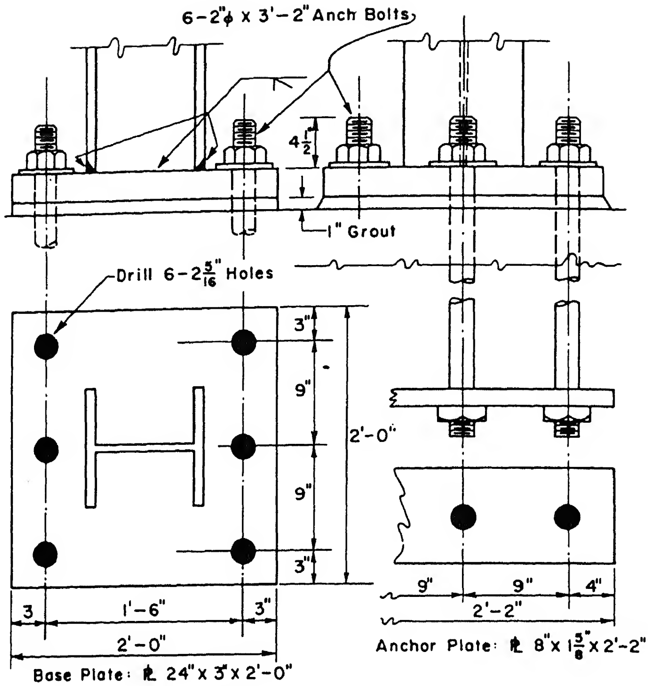
be cut. The stiffeners on the girders are not shown. The roof slab is held down by a steel spiral looped around the slab steel and welded to the purlin flange.

Girder to Column Connection. The shear force in the column web reaches a large value in the region between the girder flanges, necessitating column web reinforcing plates.



Girder To Column Conn.

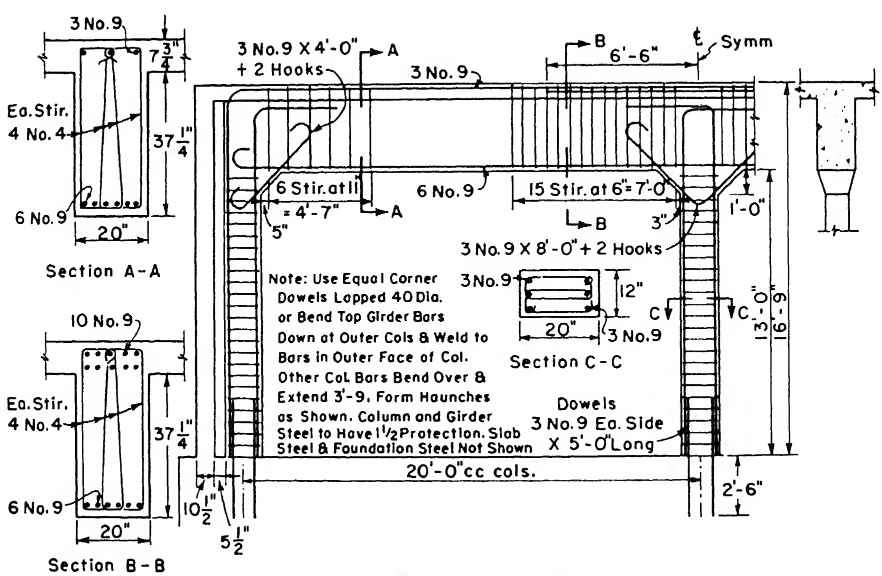
Column to Foundation Connection. The base plate is shipped attached to the column to make possible a simple connection with reliable and easily inspected shop welds.



Column to Foundation

7-54 REINFORCED CONCRETE DETAILS. Typical details are presented to suggest possible treatments in critical locations.

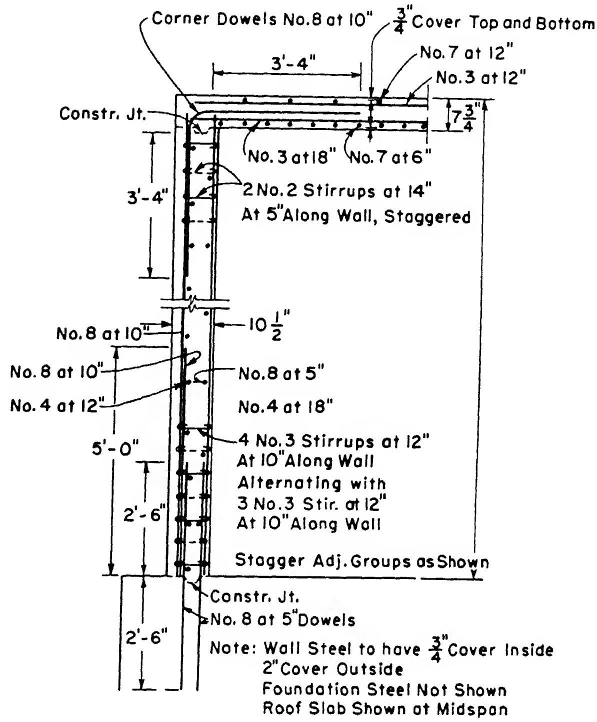
Girder and Column Steel. Bars at the intersection of the girder and



Girder and Column Steel

the outer column are shown butt-welded although dowels may be used. Dowels are shown elsewhere. The girder stirrups are bent to act as ties for bars in compression.

Wall and Roof Slab Section. Corner dowels are placed to minimize any tendency to tear out of the concrete due to either inward or outward pressure. Shear reinforcement is arranged in a staggered pattern across the face of the wall.



Wall and Roof Slab Section

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